SECTION
2.1

METHODS FOR ESTIMATING STORMWATER RUNOFF

2.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater controls. Failure to properly determine peak runoff rates can result in under –design of stormwater management systems leading to numerous adverse impacts including flooding, erosion and property damage. Over-design of these systems can result in excessive construction costs. Therefore, the designer must use good engineering judgment in preparing the hydrologic analysis for a project.

In the hydrologic analysis of a development site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for a site or drainage sub-basin; however, the following methods presented in this section have been selected to support hydrologic site analysis for the design methods and procedures included in the Manual:

- ❑ Rational Method
- ❑ SCS Unit Hydrograph Method
- ❑ USGS Regional Regression Equations
- ❑ Water Quality Treatment Volume Calculation
- ❑ Water Balance Calculations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

Table 2.1.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 2.1.1-2 provides some limitations on the use of several methods.

In general:

- The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters. Its use is limited to contributing drainage basins of ten acres or less. When using the rational method to determine detention volume, the Modified Rational Method shall be used.

- For contributing drainage areas of 100 acres or greater, the SCS Unit Hydrograph Method shall be used as a minimum. Analysis shall be based on a Type II 24-hour rainfall distribution.
- Unless stream gage data is available for the project site, use of the USGS Regional Regression Equations shall be limited to project which are located within, or adjacent to, a mapped flood hazard area.

If other hydrologic methods are to be considered and used by a design engineer, the method should first be approved by the Columbia County Engineering Services Department. Such methods shall be calibrated to local conditions and tested for accuracy and reliability. If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

MINIMUM DESIGN REQUIREMENTS

Unless noted otherwise, practices for stormwater management shall be based on a hydrologic-hydraulic analysis which considers how the stormwater management system will function with and without the proposed facilities. For such analyses, the following land use conditions shall be used:

1. For the design of the facility outlet structure, use developed land use conditions for the area within the proposed development and existing land use conditions for the upstream areas draining to the facility.

2. For any analysis of flood flows downstream of the proposed facility, use existing land use conditions for all downstream areas.

3. Auxiliary spillways (i.e. emergency spillways) for all permanent stormwater management facilities shall be designed to pass the 100-year storm event with a minimum of 6 inches of free-board. For this analysis, developed land use conditions for all areas within the analysis may be required by Columbia County.

Stormwater management facilities may include both structural and nonstructural elements. Natural swales and other natural runoff features shall be retained where practicable. Site designs shall maximize pervious areas and minimize the generation of stormwater runoff.

All stormwater management retention/detention facilities shall be designed to detain the 2, 10, 25, 50 and 100-year storm events to not exceed pre-development runoff conditions. The results of the analysis shall be included in the Stormwater Management Plan, or in a hydrologic-hydraulic study report.

Table 2.1.1-1 Applications of the Recommended Hydrologic Methods				
Method	**Manual Section**	**Rational Method**	**SCS Method**	**Water Quality Volume**
Water Quality Volume (WQ_v)	1.3			✓
Channel Protection Volume (Cp_v)	1.3		✓	
Overbank Flood Protection (Q_{p50})	1.3		✓	
Extreme Flood Protection (Q_f)	1.3		✓	
Storage Facilities	2.2		✓	
Outlet Structures	2.3		✓	
Gutter Flow and Inlets	4.2	✓		
Storm Drain Pipes	4.2	✓	✓	
Culverts	4.3	✓	✓	
Small Ditches	4.4	✓	✓	
Open Channels	4.4		✓	
Energy Dissipation	4.5		✓	

Table 2.1.1-2 Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 – 100 acres*	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
SCS[2]	0 – 2000 acres**	Method can be used for estimating peak flows and hydrographs for all design applications.
Water Quality	Limits set for each Structural Control	Method used for calculating the Water Quality Volume (WQ_v)
USGS Regional Regression	See Note 3	To be used for analysis of flood prone areas

[1] Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).

[2] There are many readily available programs (such as HEC-1) that utilize this methodology.

[3] Size limitation per *Water-Resources Investigations Report* 95-4017/

* **Modified Rational Method can be used for storage design for up to ten acres.**

** 2,000-acre upper size limit applies to single basin simplified peak flow only.

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

2.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 2.1.2-1 Constraints on Using Recommended Hydrologic Methods

Symbol	**Definition**	**Units**
A	Drainage Area	acres
B_f	Base flow	acre-feet
C	Runoff Coefficient	-
C_f	Frequency Factor	-
CN	SCS-Runoff Curve Number	-
CP_v	Channel Protection Volume	acre-feet
d	Time Interval	hours
E	Evaporation	ft
E_t	Evapotranspiration	ft
F	Pond and Swamp Adjustment Factor	-
G_h	Hydraulic Gradient	
I or i	Runoff Intensity	in/hr
I	Percent of Impervious Cover	%
I	Infiltration	ft
I_a	Initial Abstraction from Total Rainfall	in
k_h	Infiltration Rate	ft / day
L	Flow Length	ft
n	Manning Roughness Coefficient	-
O_f	Overflow	acre-feet
P	Accumulated Rainfall	in
P_2	2-year, 24-hour Rainfall	in
P_w	Wetted Perimeter	ft
PF	Peaking Factor	
Q	Rate of Runoff	cfs (or inches)
Q_d	Developed Runoff for the Design Storm	in
Q_f	Extreme Flood Protection Volume	acre-feet
Q_i	Peak Inflow Discharge	cfs
Q_o	Peak Outflow Discharge	cfs
Q_p	Peak Rate of Discharge	cfs

Q_{p25}	Overbank Flood Protection Volume	acre-feet
Q_{wq}	Water Quality Peak Rate of Discharge	cfs
q	Storm Runoff during a Time Interval	in
q_u	Unit Peak Discharge	cfs (or cfs/mi^2/inch)
R	Hydraulic Radius	ft
R_o	Runoff	acre-feet
R_v	Runoff Coefficient	
S	Ground Slope	ft / ft or %
S	Potential Maximum Retention	in
S	Slope of Hydraulic Grade Line	ft / ft
SCS	Soil Conservation Service	-
T	Channel Top Width	ft
T_L	Lag Time	hours
T_p	Time to Peak	hr
T_t	Travel Time	hours
t	Time	min
t_c	Time of Concentration	min
TIA	Total Impervious Area	%
V	Velocity	ft / s
V	Pond Volume	acre-feet
V_r	Runoff Volume	acre-feet
V_s	Storage Volume	acre-feet
WQ_v	Water Quality Volume	acre-feet

2.1.3 Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs *Depth (inches)* – Total amount of rainfall occurring during the storm duration *Intensity (inches per hour)* – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedence probability* or *return period*.

Exceedence Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1 year

Return Period – Average length of time between events that have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedence probability of 0.01 and a return period of 100 years.

Rainfall intensities for the local area are provided in Appendix A and should be used for all hydrologic analysis. The values in these tables were derived in the following way:

- Initial values were derived from TP40 (Hershfield, 1961) and HYDRO 35 (NOAA, 1977) with the 60-minute and shorter values coming from HYDRO 35.
- Intensity values for smaller than the 2-year storm were extrapolated through a series of plots.
- All values were plotted and smoothed to ensure continuity between the two different sources and to catch any errors. The values for 60 minutes and less were fit using an equation of the form:

$$\mathbf{i = a / (t + b)} \qquad \textbf{(2.1.1)}$$

where *i* is inches per hour and *t* is in minutes. *a* and *b* are fitting parameters found at the top of each of the tables in Appendix A. The tables are applicable to storm durations up to and including 1 hour. This equation allows for automated calculation of rainfall values for the Rational Method without having to look values up in tables or interpolate them from charts. The time of concentration is then substituted for **t** in Equation 2.1.1. The user can either use the values given in the tables or use the equations to calculate rainfall intensity values for times up to and including 1 hour.

Figure 2.1.3-2 shows an example Intensity-Duration-Frequency (IDF) Curve for the seven storms (1-year – 100-year). These curves are plots of the tabular values. No values are given for times less than 5 minutes.

Figure 2.1.3-3 (included as the 10-year 24-hour values from TP40) shows that the rainfall values vary south to north with generally constant values in a “V” pattern from east to west in central and south Georgia. This trend is accurate except in the far northeast corner of the state where higher elevations create an anomaly due to the orographic lifting. The anomaly does not extend south from the far northeast counties; therefore it is not correct to interpolate from this area and it should be ignored in areas outside of northeast counties.

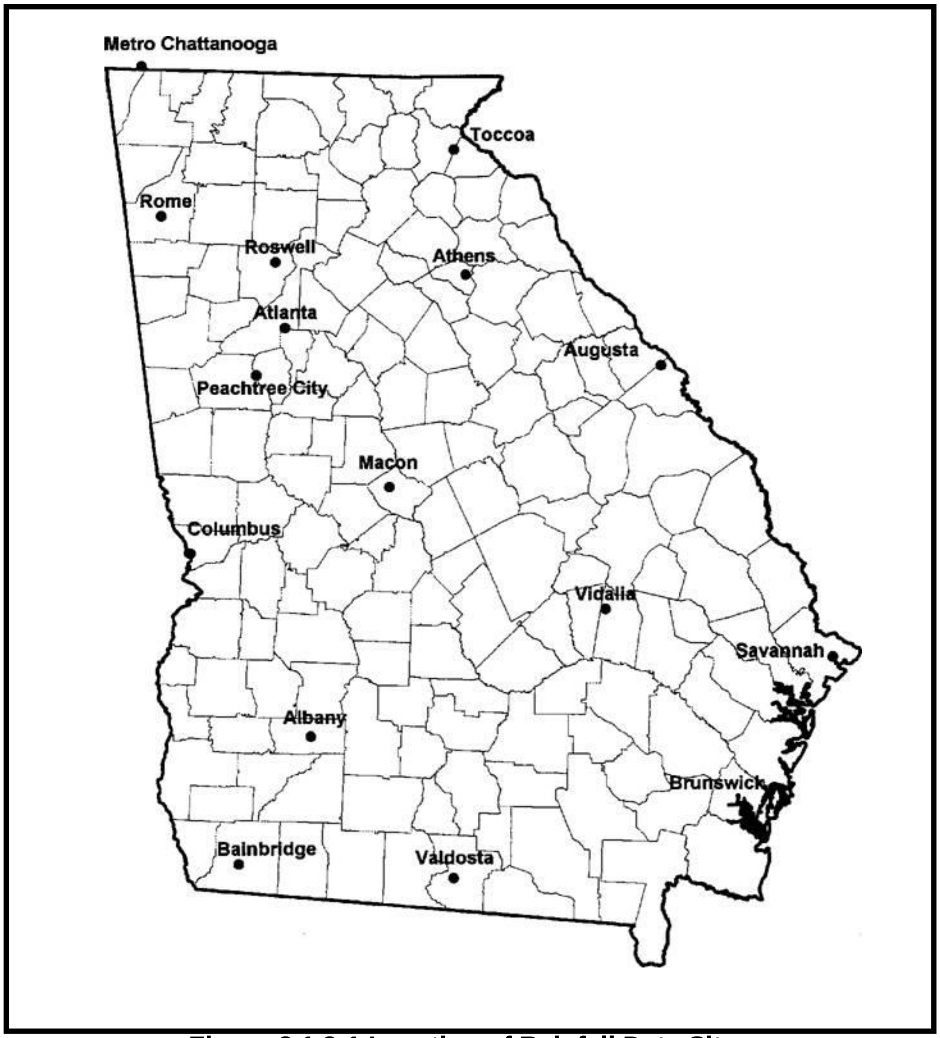


Figure 2.1.3-1 Location of Rainfall Data Sites

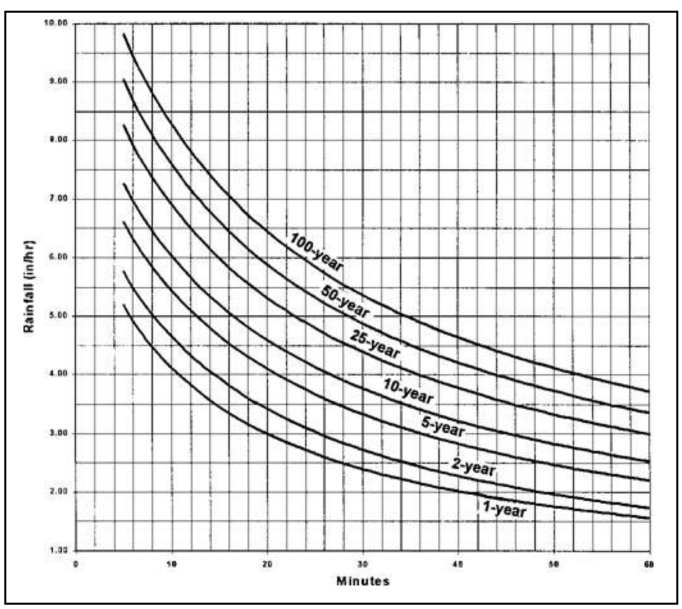


Figure 2.1.3-2 Example IDF Curve (Athens, Georgia)

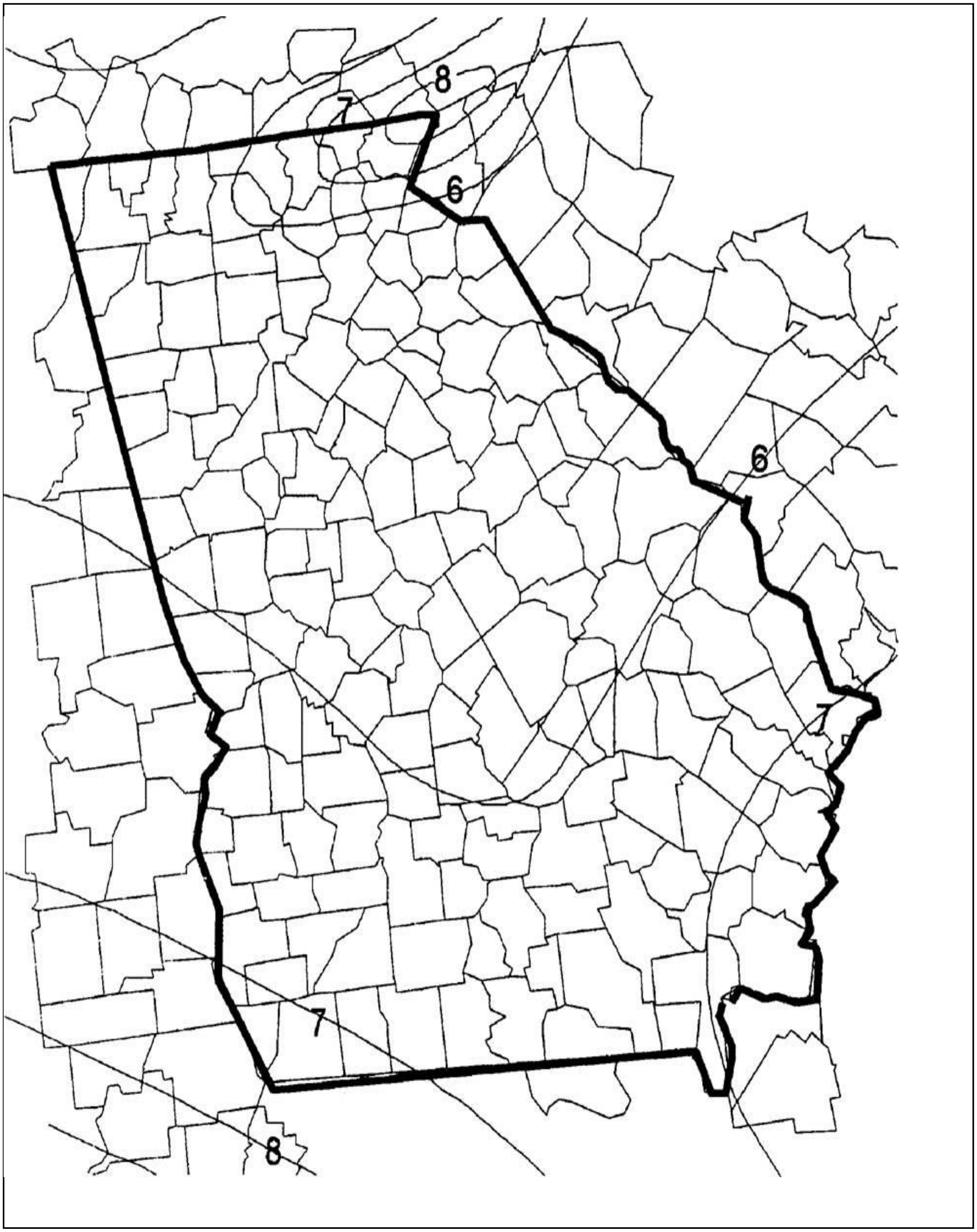


Figure 2.1.3-3 Rainfall Isohyetal Lines (10-year, 24-hour values)

2.1.4 Rational Method

2.1.4.1 Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula follows the assumption that:

- The predicted peak discharge has the same probability of occurrence (return period) as the used rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- ❑ In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- ❑ Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- ❑ The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

2.1.4.2 Application

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 10 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, due to the popularity of the Modified Rational method among Georgia practitioners for design of small detention facilities, a method has been included in Section 2.2. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in Section 2.2 corrects this deficiency in the method and can be used for detention design for drainage areas up to 10 acres.

The Rational Method should also not be used for calculating peak flows downstream of bridges, culverts or storm sewers that may act as restrictions and impact the peak rate of discharge.

2.1.4.3 Equations

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration, t_c (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

$$\mathbf{Q = CIA} \qquad \mathbf{(2.1.2)}$$

Where: Q = maximum rate of runoff (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall
I = average rainfall intensity for a duration equal to the t_c (in/hr)
A = drainage area contributing to the design location (acres)

The coefficients given in Table 2.1.4-2 are applicable for storms of 5-year to 10-year frequencies. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f. The Rational Formula now becomes:

$$\mathbf{Q = C_f CIA} \qquad \textbf{(2.1.3)}$$

The C_f values that can be used are listed in Table 2.1.4-1. The product of C_f times C shall not exceed 1.0.

Table 2.1.4-1 Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

2.1.4.4 Time of Concentration

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2.1.4-1 can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in Figure 2.1.4-2. Note: time of concentration cannot be less than 5 minutes.

Another method that can be used to determine the overland flow portion of the time of concentration is the “Kinematic Wave Nomograph” (Figure 2.1.4-3). The kinematic wave method incorporates several variables including rainfall intensity and Manning’s “n”. In using the nomograph, the engineer has two unknowns starting the computations: the time of concentration and the rainfall intensity. A value for the rainfall intensity “I” must be assumed. The travel time is determined iteratively.

If one has determined the length, slope and roughness coefficient, and selected a rainfall intensity table, the steps to use Figure 2.1.4-3 are as follows:

Step 1: Assume rainfall intensity.

Step 2: Use Figure 2.1.4-3 (or the equation given in the figure) to obtain the first estimate of time of concentration.

Step 3: Using the time of concentration obtained from Step 2, use the appropriate rainfall intensity table in Appendix A and find the rainfall intensity corresponding to the computed time of concentration. If this rainfall intensity corresponds with the assumed intensity, the problem is solved. If not, proceed to Step 4.

Step 4: Assume a new rainfall intensity that is between that assumed in Step 1 and that determined in Step 3.

Step 5: Repeat Steps 1 through 3 until there is good agreement between the assumed rainfall intensity and that obtained from the rainfall intensity tables.

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow-times that must be added as part of the overall time of concentration. When this situation is encountered, it is best to compute the confined flow-times as the first step in the overall determination of the time of concentration. This will give the designer a rough estimate of the time involved for the overland flow, which will give a better first start on the rainfall intensity assumption. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes.

Other methods and charts may be used to calculate overland flow time if approved by the County Engineer.

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

2.1.4.5 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in the rainfall tables at the end of this section, or through the use of equation 2.1.1.

2.1.4.6 Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2.1.4-2 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2.1.4-2 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

2.1.4.7 Example Problem

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

Site Data

From a topographic map and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition the following data were measured:

Average overland slope = 2.0%

Length of overland flow = 50 ft

Length of main basin channel = 2,250 ft

Slope of channel - .018 ft/ft = 1.8%

Roughness coefficient (n) of channel was estimated to be 0.090

From existing land use maps, land use for the drainage basin was estimated to be:

Residential (single family) - 80%

Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 2.1.4-2 to be 0.10.

Time of Concentration

From Figure 2.1.4-1 with an overland flow length of 50 ft, slope of 2% and a C of 0. 10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 2.1.4-2 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

$$\text{Flow Time} = \frac{\mathbf{2{,}250 feet}}{\mathbf{(3.1 ft/s)/(60 s/min)}} = 12.1 \text{ minutes}$$

and t_c = 10 + 12.1 = 22.1 min (use 22 min)

Rainfall Intensity

Using a duration equal to 22 minutes,

I_{25} (25-yr return period) = 5.06 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 2.1.4-2.

Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Weighted Runoff Coefficient***
Residential (single family)	.80	.50	.40
Graded area	.20	.30	.06

Total Weighted Runoff Coefficient = .46

*Column 3 equals column 1 multiplied by column 2.

Peak Runoff

The estimate of peak runoff for a 25-yr design storm for the given basin is:

$$\mathbf{Q_{25} = C_f CIA = (1.10)(0.46)(5.06)(23) = 59 cfs}$$

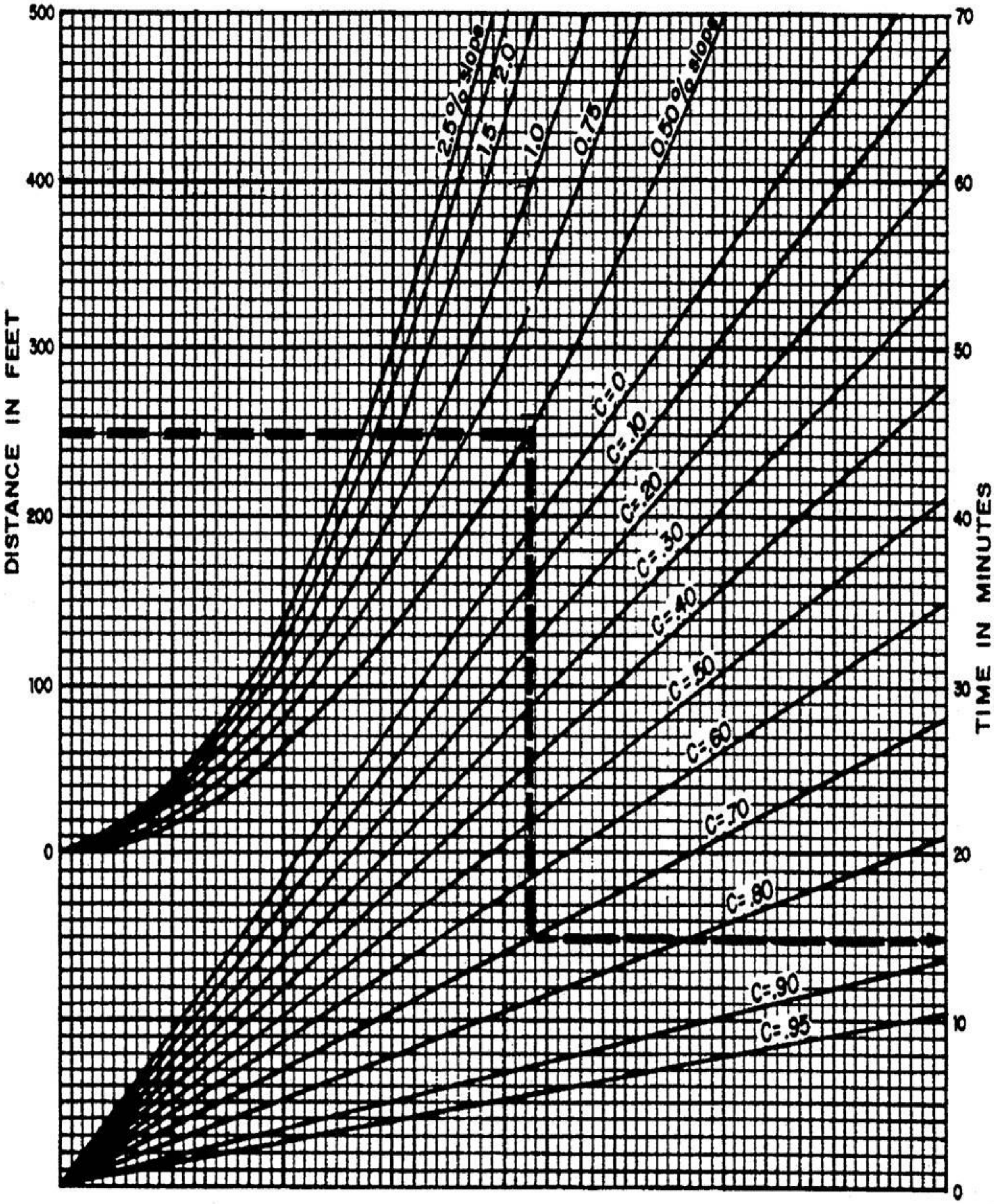
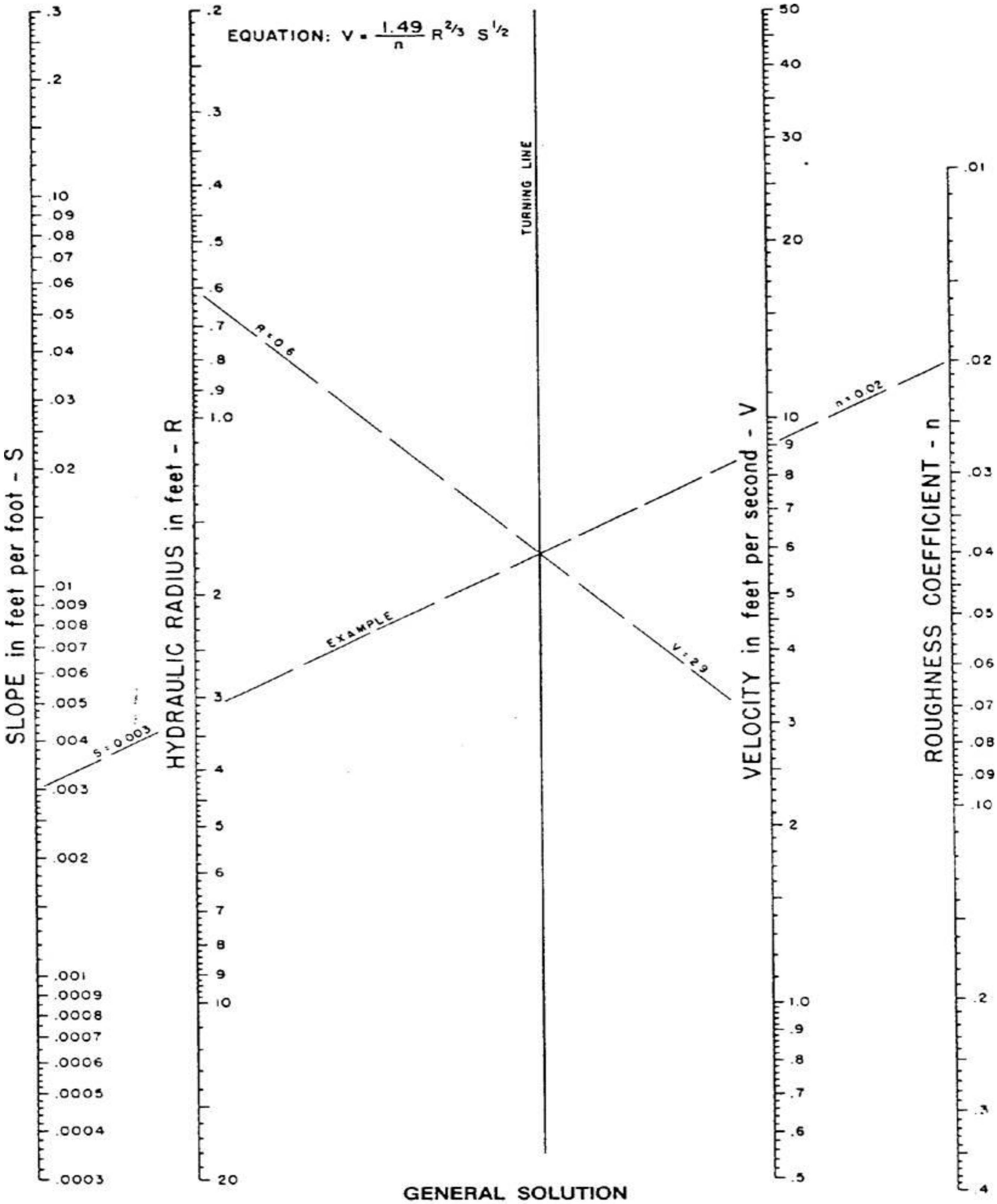


Figure 2.1.4-1 Rational Formula – Overland Time of Flow Nomograph

(Source: Airport Drainage, Federal Aviation Administration, 1965)

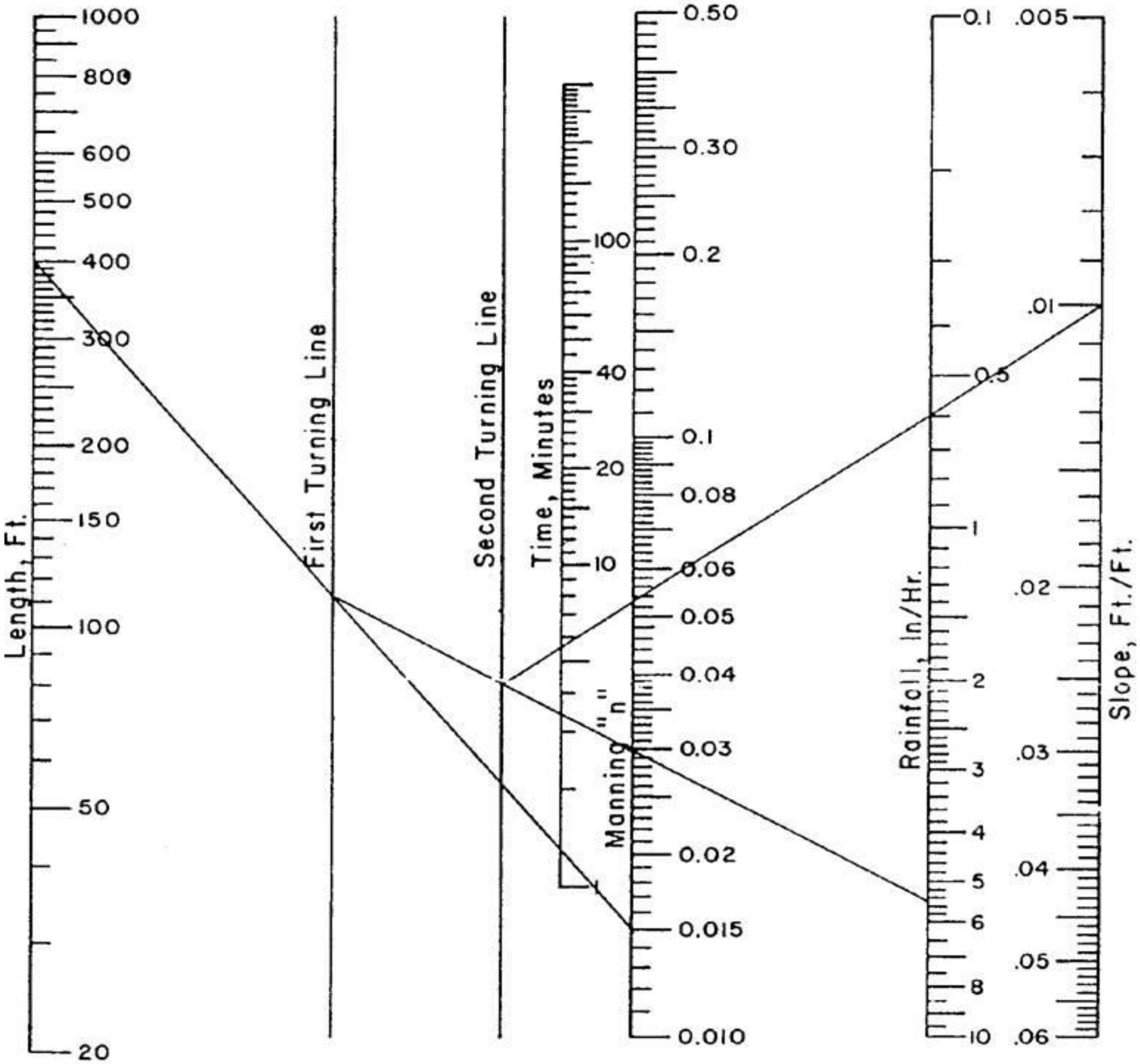


Reference: USDOT, FHWA, HDS-3 (1961).

Figure 2.1.4-2 Manning's Equation Nomograph

(Source: USDOT, FHWA, HDS-3 (1961))

Equation solved by nomograph: $t_c\,(\text{Sec}) = 56\,\dfrac{L_o^{.6}\,n^{.6}}{i^{.4}\,S_o^{.3}}$



Example:

L_o = 400 ft.
n = 0.015
i = 5.5 in./hr.
S_o = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Figure 2.1.4-3 Kinematic Wave Nomograph
(Source: Manual for Erosion and Sediment Control in Georgia, 1996)

Table 2.1.4-2 Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficients (C)
Lawns	
Sandy Soil, Flat, 2%	0.10
Sandy Soil, Average 2-7%	0.15
Sandy Soil, Steep > 7%	0.20
Clay Soil, Flat, 2%	0.17
Clay Soil, Average, 2-7%	0.22
Clay Soil, Steep, >7%	0.35
Unimproved areas (forest)	0.15
Business	
Downtown areas	0.95
Neighborhood areas	0.70
Residential	
Single-family areas	0.50
Multi-units, detached	0.60
Multi-units, attached	0.70
Suburban	0.40
Apartment Dwelling Areas	0.70
Industrial	
Light areas	0.70
Heavy areas	0.80
Parks, Cemeteries	0.25
Playgrounds	0.35
Railroad Yard Areas	0.40
Streets	
Asphalt and Concrete	0.95
Brick	0.85
Drives, Walks, and Roofs	0.95
Gravel areas	0.50
Graded or No Plant Cover	
Sandy Soil, Flat, 0 – 5%	0.30
Sandy Soil, Flat, 5 – 10%	0.40
Clayey Soil, Flat, 0 – 5%	0.50
Clayey Soil, Average, 5 – 10%	0.60

2.1.5 SCS Hydrologic Method

2.1.5.1 Introduction

The Soil Conservation Service (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the

SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

(1) Determination of curve numbers that represent different land uses within the drainage area.

(2) Calculation of time of concentration to the study point.

(3) Using the Type II rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 2.1.5-1 for the geographic boundaries for the different SCS rainfall distributions.

(4) Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

2.1.5.2 Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified method of subsection 2.1.5.7 can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

2.1.5.3 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size and slope are constant, the unit hydrograph approach assumes that there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basin concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to Columbia County is based on a storm event that has a Type II time distribution. This distributions is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 2.1.5-1).

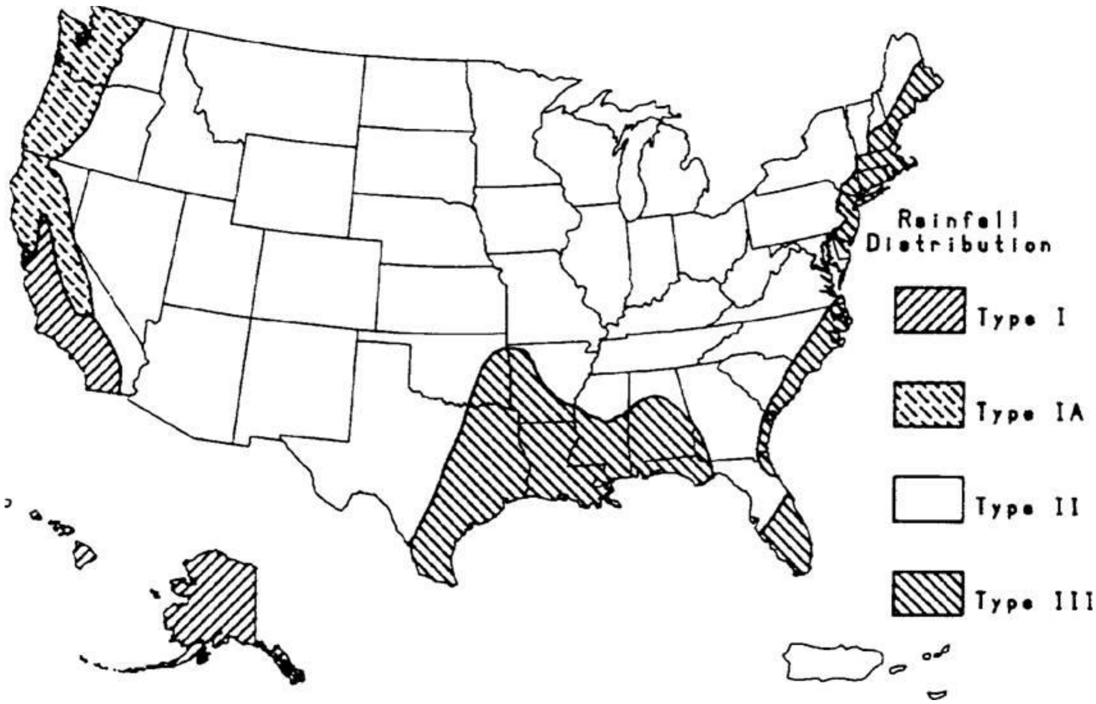


Figure 2.1.5-1 Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = \frac{(P - I_a)^2}{(P - I_a) + S}} \qquad \textbf{(2.1.4)}$$

Where: Q = accumulated direct runoff (in)

P = accumulated rainfall (potential maximum runoff) (in)

I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)

S = potential maximum soil retention (in)

An empirical relationship used in the SCS method for estimating I_a is:

$$\mathbf{I_a = 0.2S} \qquad \textbf{(2.1.5)}$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment.

Substituting 0.2S for I_a in equation 2.1.4, the equation becomes:

$$\mathbf{Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}} \qquad \textbf{(2.1.6)}$$

Where: S = 1000/CN - 10 and CN = SCS curve number

Figure 2.1.5-2 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

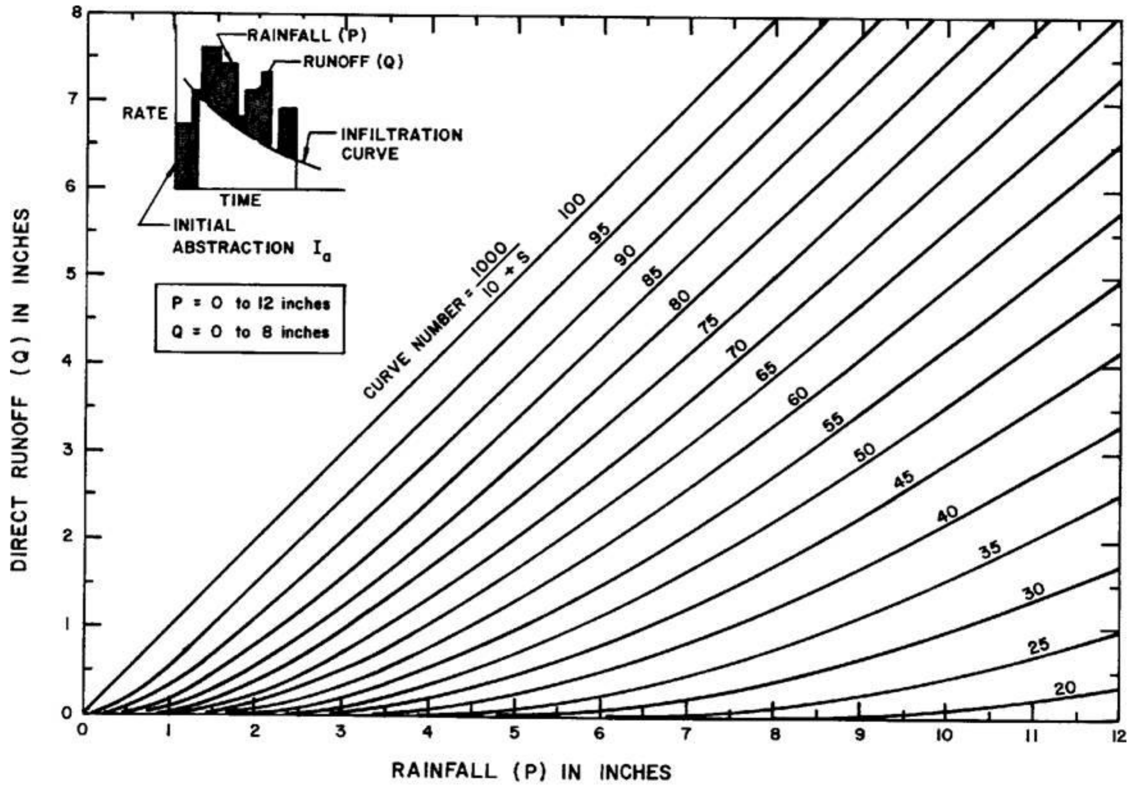


Figure 2.1.5-2
SCS Solution of the Runoff Equation
(Source: SCS, TR-55, Second Edition, June 1986)

Equation 2.1.6 can be rearranged so that the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}]} \qquad \textbf{(2.1.7)}$$

2.1.5.4 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups.

Group A — Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B — Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Georgia and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local SCS offices for use in estimating soil type.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis, except in the design of state-regulated Category I dams where AMC III may be required. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 2.1.5-1 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

Composite Curve Number Calculation Example

Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil group B	0.8	85	68
Meadow Good condition Soil group C	0.2	71	14.2

Total Weighted Curve Number = 68 + 14.2 = 82.2

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

2.1.5.5 Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2.1.5-1 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

The CNs provided in Table 2.1.5-1 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

(a) Pervious urban areas are equivalent to pasture in good hydrologic condition, and

(b) Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2.1.5-1 are not applicable, use Figure 2.1.5-3 to compute a composite CN. For example, Table 2.1.5-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2.1.5-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Table 2.1.5-1 Runoff Curve Numbers[1]

Cover description			Curve numbers for hydrologic soil groups			
Coverage Type hydrologic condition		*Average Percent impervious area*[2]	A	B	C	D
Cultivated land:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow:	good condition		30	58	71	78
Wood or forest land:	thin stand, poor cover		45	66	77	83
	good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]						
	Poor condition (grass cover <50%)		68	79	86	89
	Fair condition (grass cover 50% to 75%)		49	69	79	84
	Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:						
	Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:						
	Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
	Paved; open ditches (including right-of-way)		83	89	92	93
	Gravel (including right-of-way)		76	85	89	91
	Dirt (including right-of-way)		72	82	87	89
Urban districts:						
	Commercial and business	85%	89	92	94	95
	Industrial	72%	81	88	91	93
Residential districts by average lot size:						
	1/8 acre or less (town houses)	65%	77	85	90	92
	1/4 acre	38%	61	75	83	87
	1/3 acre	30%	57	72	81	86
	1/2 acre	25%	54	70	80	85
	1 acre	20%	51	68	79	84
	2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation)			77	86	91	94

[1]Average runoff condition, and Ia = 0.2S

[2]The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equiva

[3]CN's shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2.1.5-4 if total impervious area is less than 30% or (2) use Figure 2.1.5-3 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 2.1.5-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 2.1.5-4 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2.1.5-3) would be 68.

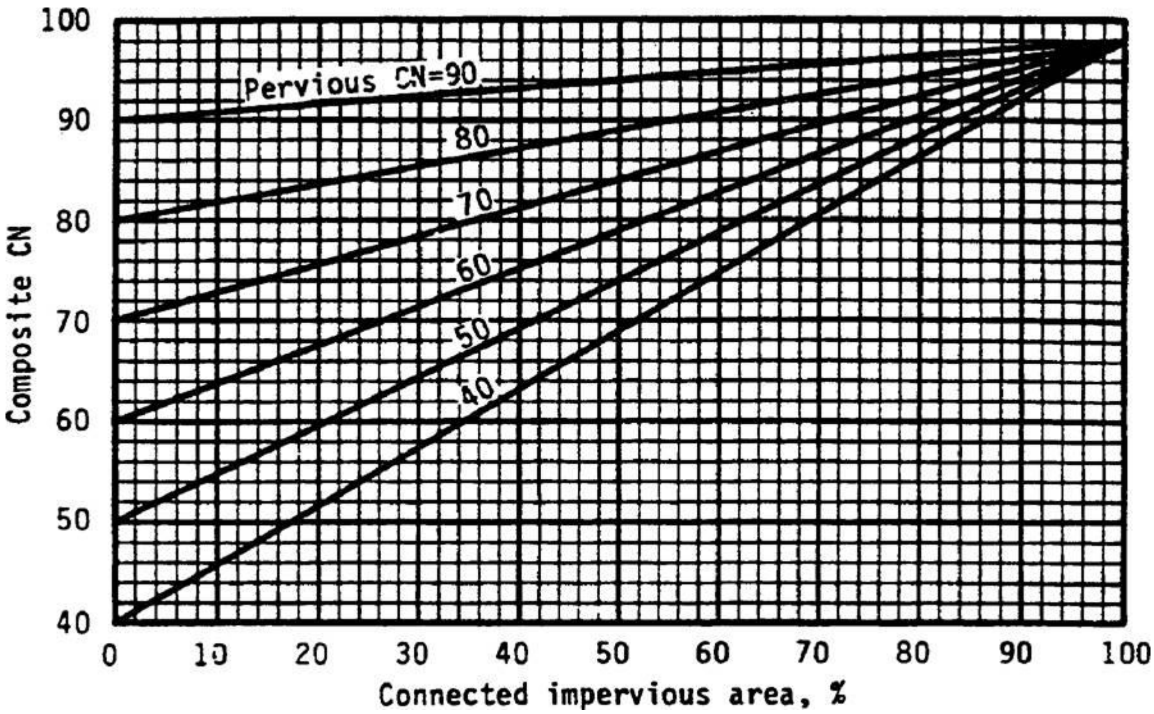


Figure 2.1.5-3 Composite CN with Connected Impervious Areas
(Source: SCS, TR-55, Second Edition, June 1986)

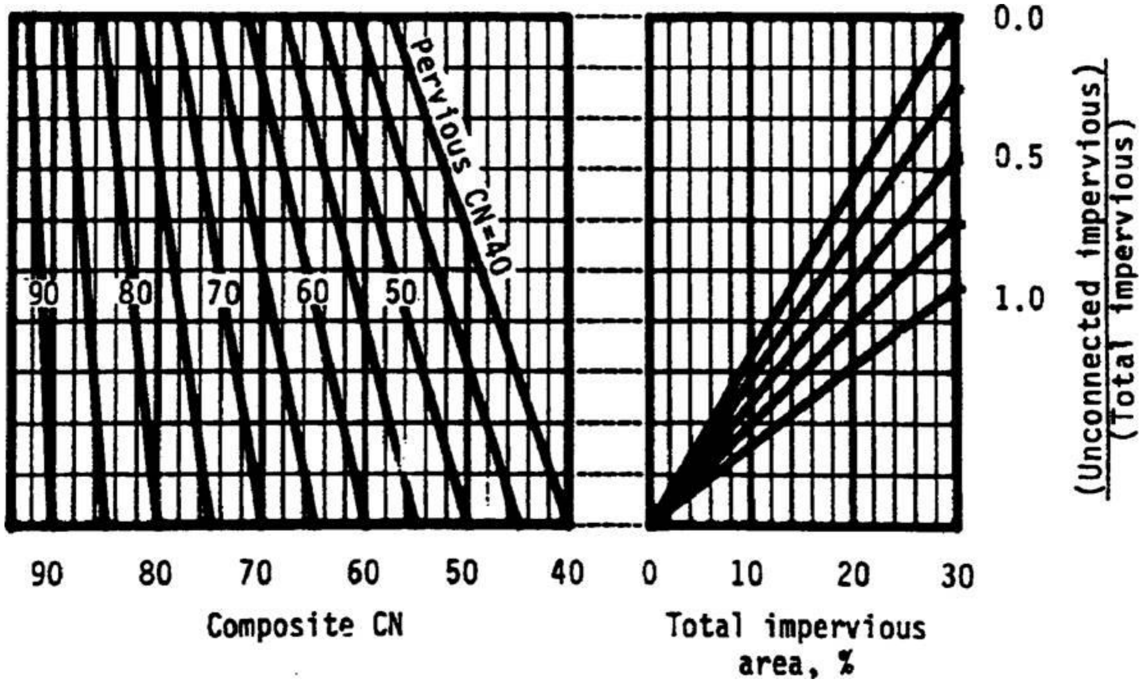


Figure 2.1.5-4 Composite CN with Unconnected Impervious Areas
(Total Impervious Area Less Than 30%)
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.6 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$\mathbf{T_t} = \frac{\mathbf{L}}{\mathbf{3600V}} \qquad \textbf{(2.1.8)}$$

Where: T_t = travel time (hr)

L = flow length (ft)

V = average velocity (ft/s)

3600 = conversion factor from seconds to hours

Sheet Flow

Sheet flow can be calculated using the following formula:

$$\mathbf{T_t} = \frac{\mathbf{0.42(nL)^{0.8}}}{\mathbf{60(P_2)^{0.5}(S)^{0.4}}} \qquad \textbf{(2.1.9)}$$

Where: T_t = travel time (hr)

n = Manning roughness coefficient (see Table 2.1.5-2)

L = flow length (ft),

P_2 = 2-year, 24-hour rainfall

S = land slope (ft/ft)

Table 2.1.5-2 Roughness Coefficients (Manning's n) for Sheet Flow[1]

Surface Description	n
Smooth Surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue Cover < 20%	0.06
Residue Cover > 20%	0.17
Grass:	
Short Grass Prairie[2]	0.15
Dense Grasses	0.24
Bermuda Grass	0.41
Range (natural)	0.13
Woods[3]	
Light Underbrush	0.40
Dense Underbrush	0.80

1 The n values are a composite of information by Engman (1986).

2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Source: SCS, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After a maximum of 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2.1.5-5, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2.1.5-5, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $$\mathbf{V = 16.13(S)^{0.5}} \qquad \mathbf{(2.1.10)}$$

Paved $$\mathbf{V = 20.33(S)^{0.5}} \qquad \mathbf{(2.1.11)}$$

Where: V = average velocity (ft/s)

S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 2.1.5-5 or equations 2.1.10 or 2.1.11, use equation 2.1.8 to estimate travel time for the shallow concentrated flow segment.

Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is $$\mathbf{V = \frac{1.49(R)^{2/3}(S)^{1/2}}{n}} \qquad \mathbf{(2.1.12)}$$

Where: V = average velocity (ft/s)

R = hydraulic radius (ft) and is equal to A/P_w

A = cross sectional flow area (ft^2)

P_w = wetted perimeter (ft)

S = slope of the hydraulic grade line (ft/ft)

n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 2.1.12, T_t for the channel segment can be estimated using equation 2.1.8.

Limitations

- Equations in this section should not be used for sheet flow longer than 300 feet.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

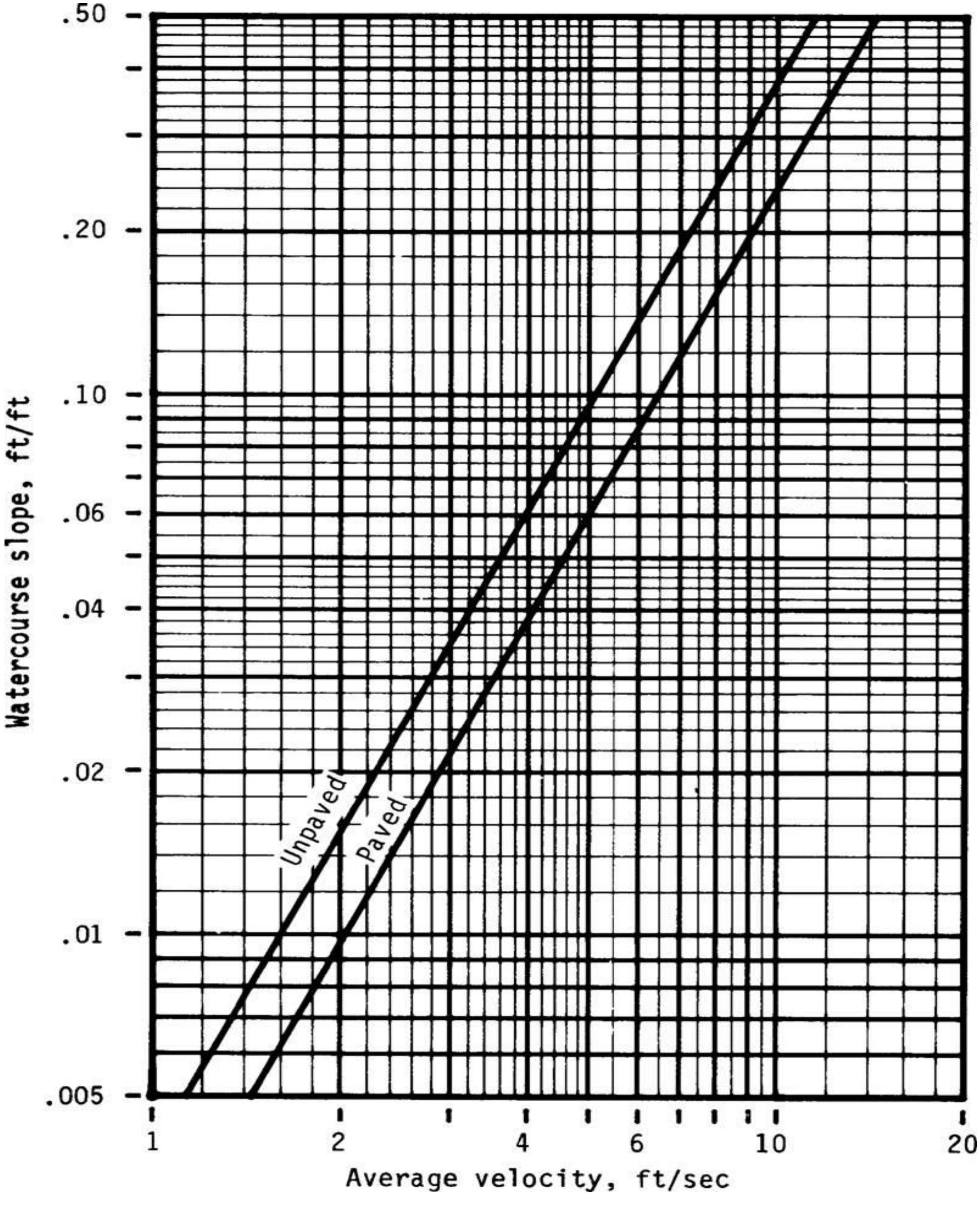


Figure 2.1.5-5 Average Velocities – Shallow Concentrated Flow
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.7 Simplified SCS Peak Runoff Rate Estimation

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses that can be described by a single CN value. The peak discharge equation is:

$$\mathbf{Q_p = q_u A Q F_p} \qquad \textbf{(2.1.13)}$$

Where: Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

The input requirements for this method are as follows:

- t_c – hours
- Drainage area – mi^2
- Type II or type III rainfall distribution
- 24-hour design rainfall
- CN value
- Pond and Swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the t_c computation, an adjustment is needed.)

Computations for the peak discharge method proceed as follows:

(1) The 24-hour rainfall depth is determined from the rainfall tables in Appendix A for the selected location and return frequency.

(2) The runoff curve number, CN, is estimated from Table 2.1.5-1 and direct runoff, Q_p, is calculated using equation 2.1.13.

(3) The CN value is used to determine the initial abstraction, I_a, from Table 2.1.5-3, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).

(4) The watershed time of concentration is computed using the procedures in subsection 2.1.5.6 and is used with the ratio I_a/P to obtain the unit peak discharge, q_{up}, from Figure 2.1.5-6 for the Type II rainfall distribution and Figure 2.1.5-7 for the Type III rainfall distribution. If the ratio I_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figures 2.1.5-6 and 2.1.5-7 are based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used.

(5) The pond and swamp adjustment factor, F_p, is estimated from below:

Pond and Swamp Areas (%*)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

(6) The peak runoff rate is computed using equation 2.1.13.

Table 2.1.5-3 I_a Values for Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Source: SCS, TR-55, Second Edition, June 1986

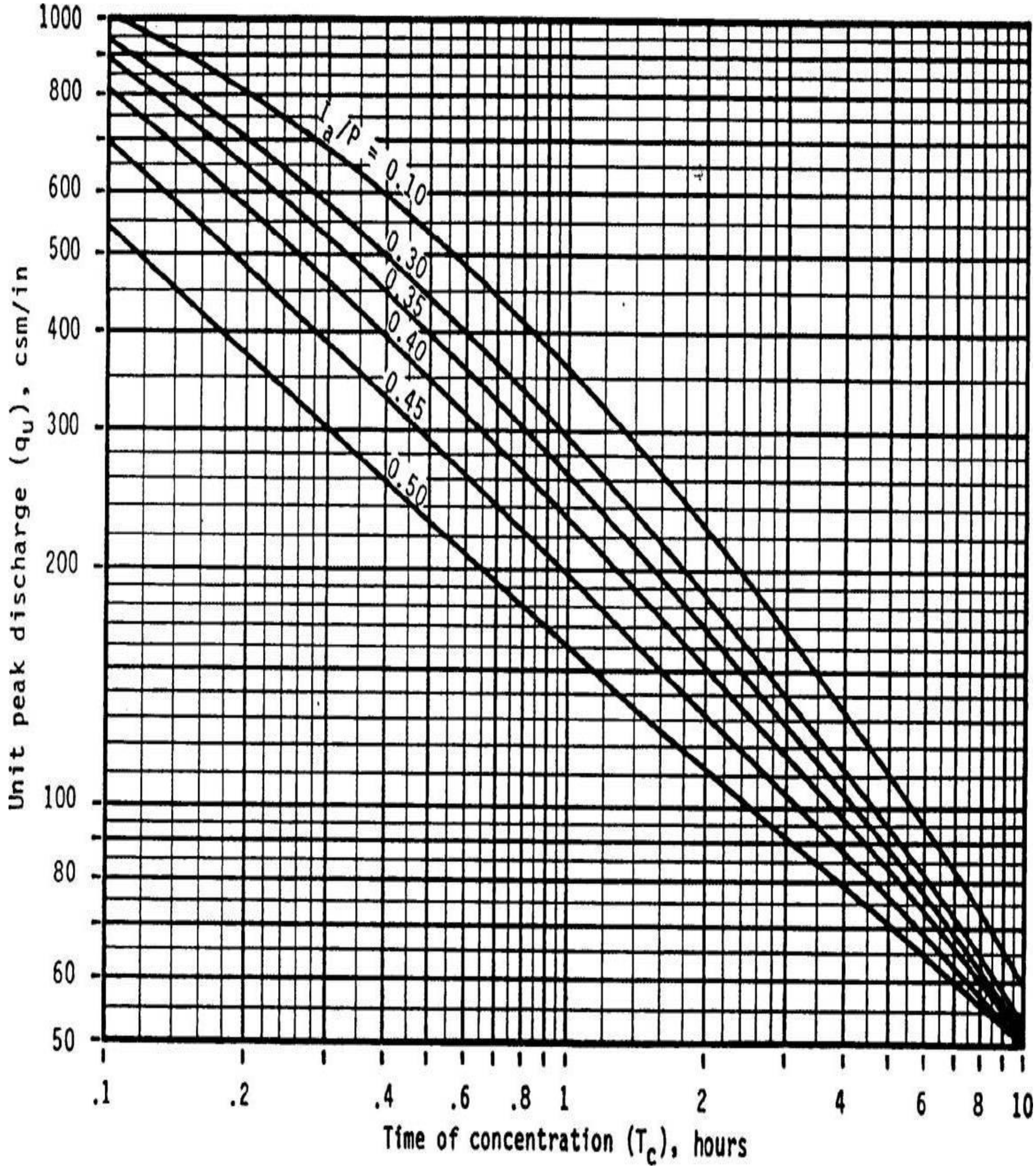


Figure 2.1.5-6
SCS Type II Unit peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

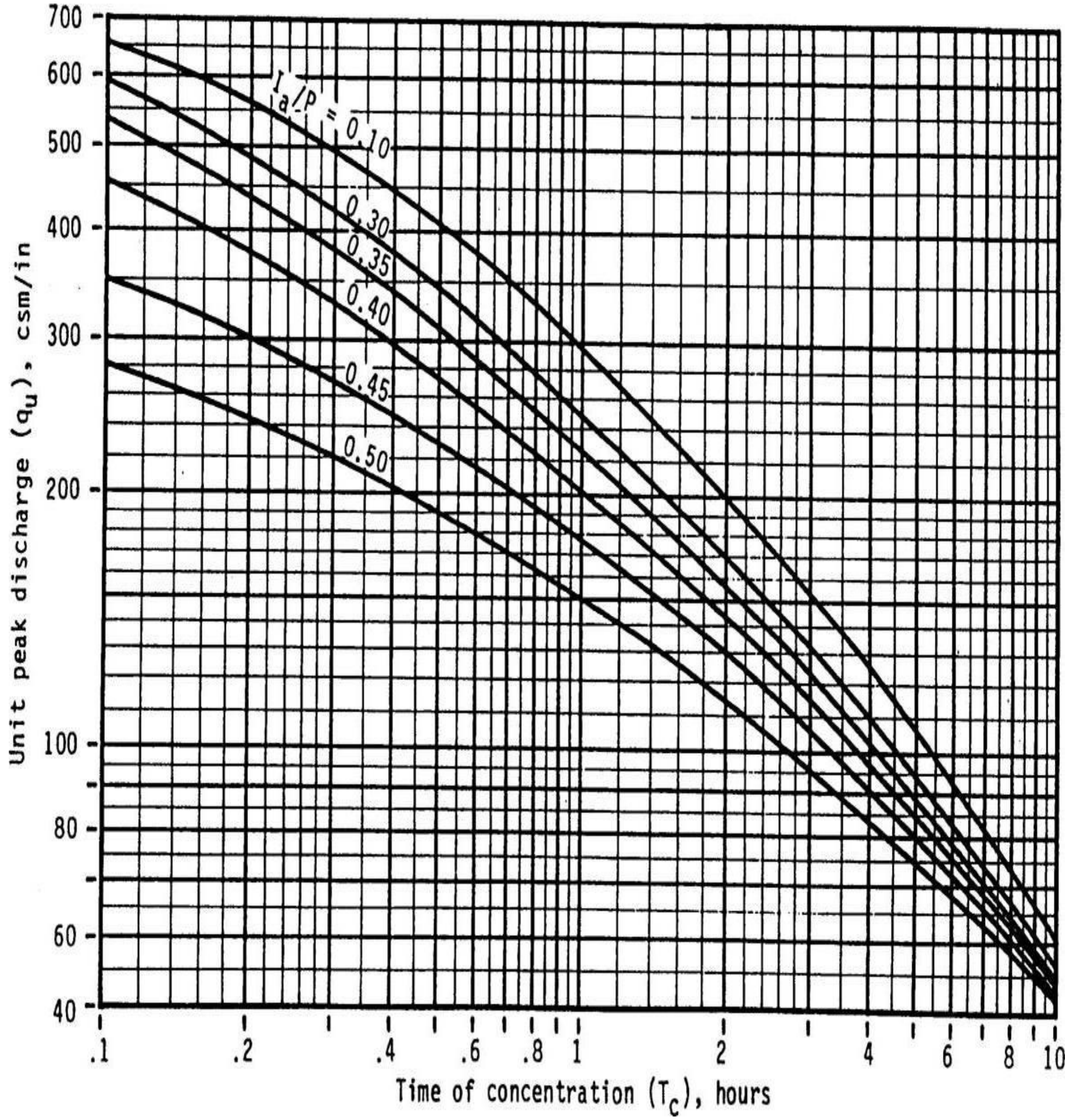


Figure 2.1.5-7
SCS Type III Unit peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.8 Example Problem 1

Compute the 100-year peak discharge for a 50-acre wooded watershed which will be developed as follows:

- Forest land - good cover (hydrologic soil group B) = 10 ac
- Forest land - good cover (hydrologic soil group C) = 10 ac
- ⅓ acre residential (hydrologic soil group B) = 20 ac
- Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

Computations

(1) Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 7.92 inches (.33 in/hr x 24 hours).
- The 1-year, 24-hour rainfall is 3.36 inches (.14 in/hr x 24 hours).
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	0.2	55	11.0
2	10 ac.	0.2	70	14.0
3	20 ac.	0.4	72	28.8
4	10 ac.	0.2	91	18.2
Total	50 ac.	1.00		72.0

* from Equation 2.1.6, Q (100-year) = 4.6 inches Q_d (1-year developed) = 1.0 inches

(2) Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	0.24	40	2.0
2	Shallow channel	750	1.7
3	Main channel*	1100	0.5

* For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from equation 2.1.9 with P2 = 4.08 inches (0.17 x 24)

$$\mathbf{T_t = [0.42(0.24 \times 40)^{0.8}] / [(4.08)^{0.5} (.020)^{0.4}] = 6.07 \text{ minutes}}$$

Segment 2 - Travel time from Figure 2.1.5-5 or equation 2.1.10

$\mathbf{V = 2.1 \text{ ft / sec}}$ (from equation 2. 1.10)

$$\mathbf{T_t = 750 / 60(2.1) = 5.95 \text{ minutes}}$$

Segment 3 - Using equation 2.1.12

$$\mathbf{V = (1.49 / 0.06)(1.43)^{0.67}(0.005)^{0.5} = 2.23 \text{ ft / sec}}$$

$$\mathbf{T_t = 1100 / 60(2.23) = 8.22 \text{ minutes}}$$

$$\mathbf{t_c = 6.07 + 5.95 + 8.22 = 20.24 \text{ minutes } (0.34 \text{ hours})}$$

(3) Calculate I_a/P for C_n = 72 (Table 2.1.5-1), I_a = .778 (Table 2.1.5-3)

$\mathbf{I_a / P = (0.778 / 7.92) = 0.098}$ (Note: Use I_a/P = .10 to facilitate use of Figure 2.1.5-6. Straight line interpolation could also be used.)

(4) Unit discharge q_u (100-year) from Figure 2.1.5-6 = 650 csm/in, q_u (1-year) = 580 csm/in

(5) Calculate peak discharge with F_p = 1 using equation 2.1.13

$$\mathbf{Q_{100} = 650(50 / 640)(4.6)(1) = 234 \text{ cfs}}$$

2.1.5.9 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas. The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat areas. Referring to Figure 2.1.6-1, which shows the different hydrologic regions developed by the USGS for the state of Georgia, Region 3 represents the primary region of the state where modification of the peaking factor from 484 to 300 is most often warranted if the individual watershed possesses flat terrain.

As a result of hydrologic/hydraulic studies completed in the development of this Manual, the following are recommendations related to the use of different peaking factors:

- The SCS method can be used without modification (peaking factor left at 484) in Regions 1, 2 and 4 generally when performing modeling analysis.
- The SCS method can be modified in that a peaking factor of 300 can be used for modeling generally in Region 3 when watersheds are flat and have significant storage in the overbanks. These watersheds would be characterized by:
    - ❑ Mild Slopes (less than 2% slope)
    - ❑ Significant surface storage throughout the watershed in the form of standing water during storm events or inefficient drainage systems

The SCS method can be similarly adjusted for any watershed that has flow and storage characteristics similar to a typical Region 3 stream

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand. For that reason only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

(1) Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in subsection 2.1.5.3 is used. This storm is recommended for use in Georgia.

(2) Development of curve numbers and lag times for the watershed using the methods described in subsections 2.1.5.4, 2.1.5.5, and 2.1.5.6.

(3) Development of a unit hydrograph from either the standard (peaking factor of 484) or coastal area (peaking factor of 300) dimensionless unit hydrographs. See discussion below.

(4) Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 2.1.6).

(5) Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called “convolution”).

(6) Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 300, Figure 2.1.5-8 and Table 2.1.5-4 have been developed. The unit hydrograph is used in the same way as the unit hydrograph with a peaking factor of 484.

The procedure to develop a unit hydrograph from the dimensionless unit hydrographs in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$\mathbf{q_u = (PF\ A)/(T_p)} \qquad \textbf{(2.1.14)}$$

Where: q_u = unit hydrograph peak rate of discharge (cfs)

PF = peaking factor (either 484 or 300)

A = area (mi^2)

d = rainfall time increment (hr)

T_p = time to peak = $d/2 + 0.6\ T_c$ (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrographs for 484 and 300 can be approximated by the equation:

$$\mathbf{\frac{q}{q_u} = \left[\frac{t}{T_p} e^{\left(1-\frac{t}{T_p}\right)}\right]^{x}} \qquad \textbf{(2.1.15)}$$

Where X is 3.79 for the PF=484 unit hydrograph and 1.50 for the PF=300 unit hydrograph.

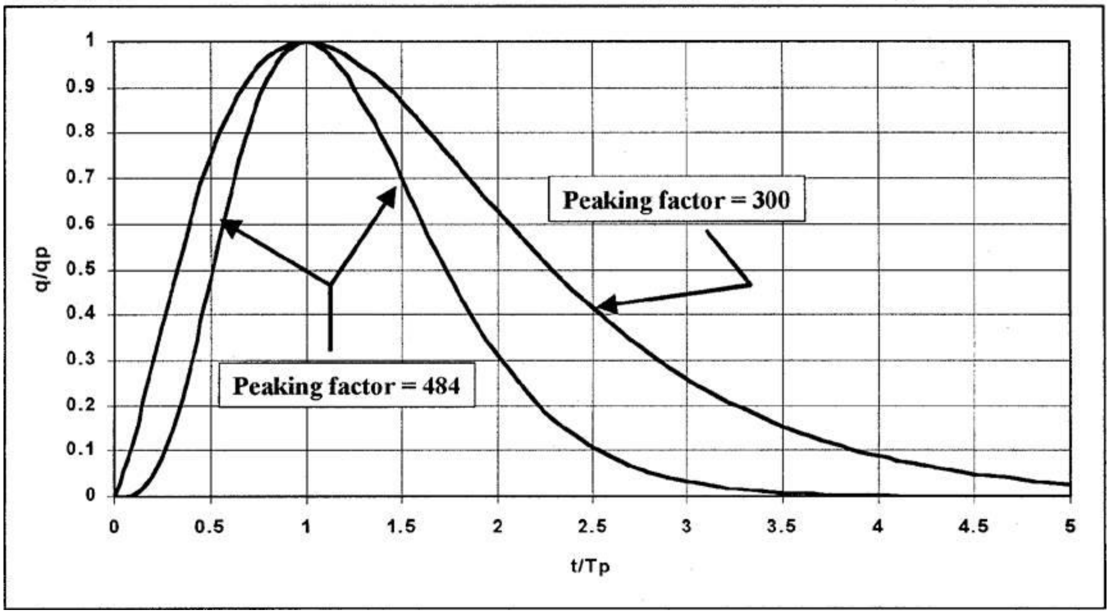


Figure 2.1.5-8 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 2.1.5-4 Dimensionless Unit Hydrographs

	484		300	
t/Tt	**q/qu**	**Q/Qp**	**q/qu**	**Q/Qp**
0.0	0.0	0.0	0.0	0.0
0.1	0.005	0.000	0.122	0.006
0.2	0.046	0.004	0.296	0.019
0.3	1.480	0.015	0.469	0.041
0.4	0.301	0.038	0.622	0.070
0.5	0.481	0.075	0.748	0.105
0.6	0.657	0.125	0.847	0.144
0.7	0.807	0.186	0.918	0.186
0.8	0.916	0.255	0.966	0.231
0.9	0.980	0.330	0.992	0.277
1.0	1.000	0.406	1.000	0.324
1.1	0.982	0.481	0.993	0.370
1.2	0.935	0.552	0.974	0.415
1.3	0.867	0.618	0.945	0.459
1.4	0.786	0.677	0.909	0.501
1.5	0.699	0.730	0.868	0.541
1.6	0.611	0.777	0.823	0.579
1.7	0.526	0.817	0.775	0.615
1.8	0.447	0.851	0.727	0.649
1.9	0.376	0.879	0.678	0.680
2.0	0.312	0.903	0.631	0.710
2.1	0.257	0.923	0.584	0.737
2.2	0.210	0.939	0.539	0.762
2.3	0.170	0.951	0.496	0.785
2.4	0.137	0.962	0.455	0.806

Table 2.1.5-4 Dimensionless Unit Hydrographs (continued)

	484		300	
t/Tt	q/qu	Q/Qp	q/qu	Q/Qp
2.5	0.109	0.970	0.416	0.825
2.6	0.087	0.977	0.380	0.843
2.7	0.069	0.982	0.346	0.859
2.8	0.054	0.986	0.314	0.873
2.9	0.042	0.989	0.285	0.886
3.0	0.033	0.992	0.258	0.898
3.1	0.025	0.994	0.233	0.909
3.2	0.020	0.995	0.211	0.919
3.3	0.015	0.996	0.190	0.928
3.4	0.012	0.997	0.171	0.936
3.5	0.009	0.998	0.153	0.943
3.6	0.007	0.998	0.138	0.949
3.7	0.005	0.999	0.124	0.955
3.8	0.004	0.999	0.111	0.960
3.9	0.003	0.999	0.099	0.965
4.0	0.002	1.000	0.089	0.969
4.1			0.079	0.972
4.2			0.071	0.976
4.3			0.063	0.979
4.4			0.056	0.981
4.5			0.050	0.984
4.6			0.044	0.986
4.7			0.039	0.987
4.8			0.035	0.989
4.9			0.031	0.990
5.0			0.028	0.992
5.1			0.024	0.993
5.2			0.022	0.994
5.3			0.019	0.995
5.4			0.017	0.996
5.5			0.015	0.996
5.6			0.013	0.997
5.7			0.012	0.997
5.8			0.010	0.998
5.9			0.009	0.998
6.0			0.008	0.999
6.1			0.007	0.999
6.2			0.006	0.999
6.3			0.006	1.000

2.1.5.10 Example Problem 2

Compute the unit hydrograph for the 50-acre wooded watershed in example 2.1.5.8.

Computations

(1) Calculate T_p and time increment

The time of concentration (T_c) is calculated to be 20.24 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

$$\mathbf{T_p = d/2 + 0.6T_c = 3/2 + 0.6T_c = 3/2 + 0.6 \times 20.24 = 13.64\ minutes\ (0.227\ hrs)}$$

(2) Calculate q_{pu}

$$\mathbf{q_{pu} = PF\,A/T_p = (484 \times 50/640)/(0.227) = 166\ cfs}$$

For a PF of 300 q_{pu} would be:

$$\mathbf{q_{pu} = PF\,A/T_p = (300 \times 50/640)/(0.227) = 103\ cfs}$$

(3) Calculate unit hydrograph for both 484 and 300.

Based on spreadsheet calculations using equations 2.1.14 and 2.1.15, the table below has been derived.

Time		484		300	
t/Tp	time (min)	q/qpu	Q	q/qpu	q
0	0	0	0.00	0	0.00
0.22	3.0	0.06	10.26	0.33	34.18
0.44	6.0	0.37	61.74	0.68	69.60
0.66	9.0	0.75	124.79	0.89	91.99
0.88	12.0	0.97	161.37	0.99	101.85
1.00	13.64	1.00	166.00	1.00	103.00
1.10	15.0	0.98	163.39	0.99	102.35
1.32	18.0	0.85	141.70	0.94	96.74
1.54	21.0	0.66	110.45	0.85	87.64
1.76	24.0	0.48	79.61	0.75	76.98
1.98	27.0	0.33	54.06	0.64	66.03
2.20	30.0	0.21	35.02	0.54	55.59
2.42	33.0	0.13	21.84	0.45	46.10
2.64	36.0	0.08	13.19	0.37	37.76
2.86	39.0	0.05	7.77	0.30	30.60
3.08	42.0	0.03	4.47	0.24	24.58
3.30	45.0	0.02	2.52	0.19	19.60
3.52	48.0	0.01	1.40	0.15	15.52
3.74	51.0	0.00	0.76	0.12	12.21
3.96	54.0	0.00	0.41	0.09	9.57
4.18	57.0	0.00	0.22	0.07	7.46
4.40	60.0	0.00	0.12	0.06	5.79
4.62	63.0	0.00	0.06	0.04	4.48
4.84	66.0	0.00	0.03	0.03	3.45
5.06	69.0	0.00	0.02	0.03	2.65
5.28	72.0	0.00	0.01	0.02	2.03

Time		484		300	
t/Tp	time (min)	q/qpu	Q	q/qpu	q
5.50	75.0	0.00	0.00	0.02	1.55
5.72	78.0			0.01	1.18
5.94	81.0			0.01	0.90
6.16	84.0			0.01	0.68
6.38	87.0			0.01	0.52
6.60	90.0			0.00	0.39
6.82	93.0			0.00	0.30
7.04	96.0			0.00	0.22
7.26	99.0			0.00	0.17
7.48	102.0			0.00	0.13
7.70	105.0			0.00	0.09
7.92	108.0			0.00	0.07
8.14	111.0			0.00	0.05
8.36	114.0			0.00	0.04
8.58	117.0			0.00	0.03
8.80	120.0			0.00	0.02
9.01	123.0			0.00	0.02
9.23	126.0			0.00	0.01
9.45	129.0			0.00	0.01
9.67	132.0			0.00	0.01
9.89	135.0			0.00	0.01
10.11	138.0			0.00	0.00

Physical Characteristics	Minimum	Maximum	Units
A - Drainage Area	0.04	19.1	mi^2
TIA - Total Impervious Area	1.00	62	percent

Region 4 (urban):

$$\mathbf{T_L = 6.10A^{0.35}TIA^{-0.22}S^{-0.31}} \quad \textbf{(2.1.19)}$$

Where: T_L = lag time (hours)

A = drainage area (mi^2)

S = main channel slope (ft/mi)

TIA = total impervious area (percent)

Using these lag time equations and the dimensionless hydrograph, a runoff hydrograph can be determined after the peak discharge is calculated.

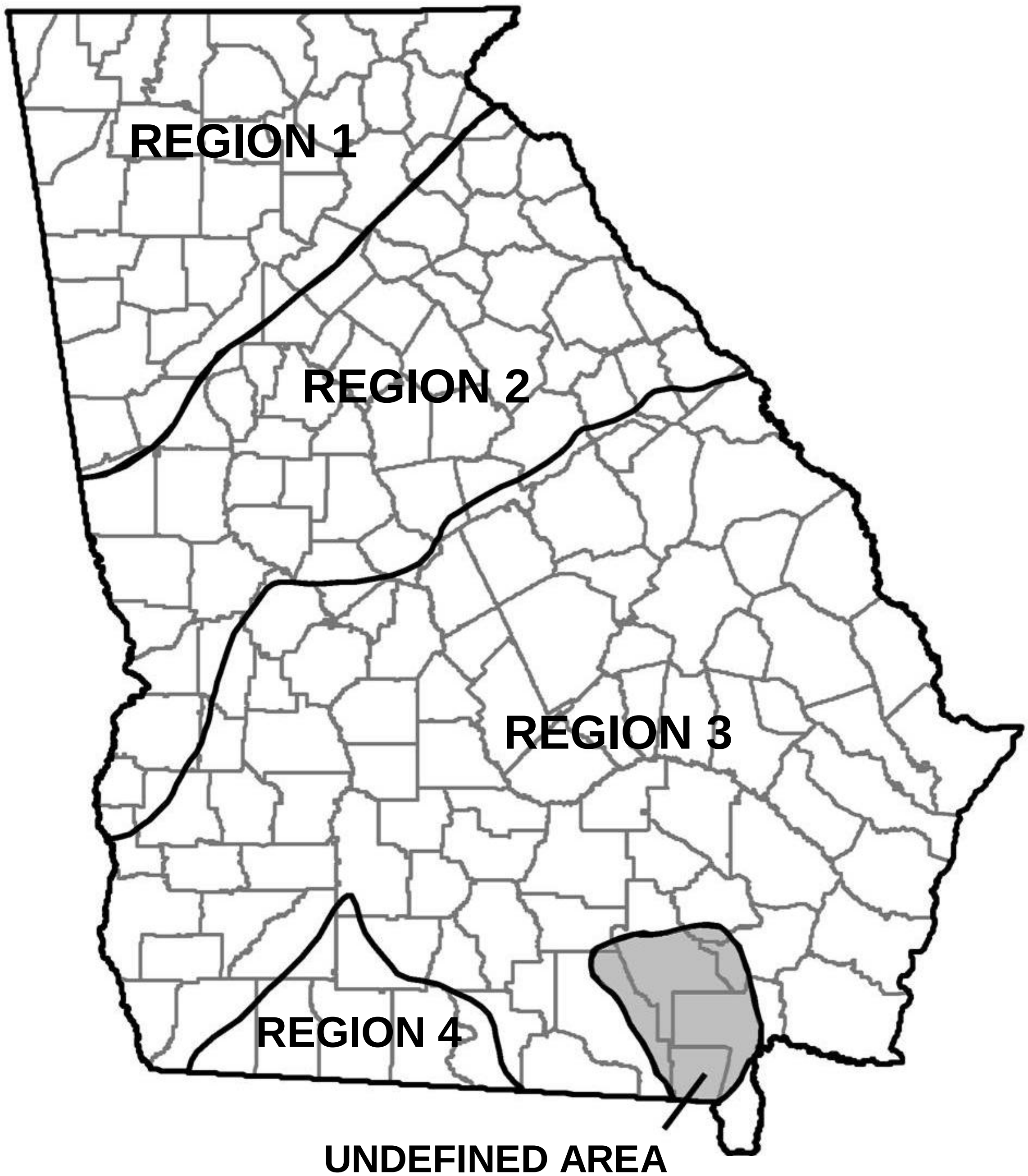


Figure 2.1.6-1 USGS Hydrologic Regions in Georgia

(Source: USGS, 1994)

2.1.6.1 Hydrograph Limitations

Following are the limitations of the variables within the lag time equations. The lag time equation should not be used for drainage areas that have physical characteristics outside the limits listed below:Regression

Physical Characteristics	Minimum	Maximum	Units
North of the Fall Line (rural)			
A - Drainage Area	0.3	500	mi^2
S - Main Channel Slope	5.0	200	feet per mile
South of the Fall Line (rural)			
A - Drainage Area	0.2	500	mi^2
S - Main Channel Slope	1.3	60	feet per mile
Regions 1, 2 & 3 (urban)			
A - Drainage Area	0.04	19.1	mi^2
S - Main Channel Slope	9.4	772.0	feet per mile
TIA - Total Impervious	1.0	61.6	percent
Region 4 (urban)			
A - Drainage Area	0.12	2.9	mi^2
S - Main Channel Slope	19.4	110.0	feet per mile
TIA - Total Impervious	6.1	42.4	percent

2.1.6.2 Rural (or Undeveloped) Basins

The USGS has recently revised the equation for estimating peak discharges for rural basins, as seen in Table 2.1.6-1. For a complete discussion of the development of these equations consult the USGS publication *Techniques for Estimating Magnitude and Frequency of Floods in Rural Basins of Georgia, Water-Resources Investigations Report 93-4016*.

2.1.6.3 Rural (or Undeveloped) Basin Limitations

Following are the limitations associated with the rural basin equations given above:

Physical Characteristics	Minimum	Maximum	Units
Region 1 - A - Drainage Area	0.17	730	mi^2
Region 2 - A - Drainage Area	0.10	3,000	mi^2
Region 3 - A - Drainage Area	0.14	3,000	mi^2
Region 4 - A - Drainage Area	0.25	2,000	mi^2

Table 2.1.6-1 USGS Rural Peak Equations[1]

Frequency	Equations Region 1	Equations Region 2
Q_2	$207A^{0.654}$	$182A^{0.622}$
Q_5	$357A^{0.632}$	$311\ A^{0.616}$
Q_{10}	$482A^{0.619}$	$411\ A^{0.613}$
Q_{25}	$666A^{0.605}$	$552A^{0.610}$
Q_{50}	$827A^{0.595}$	$669A^{0.607}$
Q_{100}	$1010A^{0.584}$	$794A^{0.605}$
Q_{200}	$1220A^{0.575}$	$931A^{0.603}$
Q_{500}	$1530A^{0.563}$	$1130A^{0.601}$

Frequency	Equations Region 3	Equations Region 4
Q_2	$76A^{0.620}$	$142A^{0.591}$
Q_5	$133A^{0.620}$	$288A^{0.589}$
Q_{10}	$176A^{0.621}$	$410A^{0.591}$
Q_{25}	$237A^{0.623}$	$591A^{0.595}$
Q_{50}	$287A^{0.625}$	$748A^{0.599}$
Q_{100}	$340A^{0.627}$	$926A^{0.602}$
Q_{200}	$396A^{0.629}$	$1120A^{0.606}$
Q_{500}	$474A^{0.632}$	$1420A^{0.611}$

A - Drainage Area in mi^2

[1] For estimating discharges for a specific recurrence interval at sites where gaged data are available from the USGS, follow procedures outlined on pages 16 and 17 in the USGS publication *Techniques for Estimating Magnitude and Frequency of Floods in Rura*

Source: USGS, 1993

2.1.6.4 Example Problem

For the 100-year flood, calculate the peak discharge for rural and developed conditions for the following drainage area located in Region 1 in the Atlanta metro area. For the developed conditions, develop the flood hydrograph for this drainage area.

- Drainage Area = 175 acres = 0.273 mi^2
- Main Channel Slope = 117 ft/mi
- Total Impervious Area (TIA) = 32%

Peak Discharge Calculations

100-year Rural Peak Discharge:

$$\mathbf{Q_{100} = 1010A^{0.584} = 1010(0.273)^{0.584} = 473\ cfs}$$

100-year Developed Peak Flow:

$$\mathbf{Q_{100} = 762A^{0.69}(32)^{0.17}}$$

$$\mathbf{Q_{100} = 762(0.273)^{0.69}(32)^{0.17} = 561\ cfs}$$

Flood Hydrograph Calculations

Lag Time Calculations

$$\mathbf{T_L = 7.86A^{0.35}TIA^{-0.22}S^{-0.31} = 7.86(0.273)^{0.35}(32)^{-0.22}(117)^{-0.31} = 0.53\ hours}$$

Hydrograph Calculations

Coordinates for the flood hydrograph are given in Table 2.1.6-2 on the next page.

Table 2.1.6-2 Flood Hydrograph

Time Ratio (t/T_t)	Time (t) Hours	Discharge Ratio (Q/Q_p)	Discharge (cfs)
0.25	0.13	0.12	67
0.30	0.16	0.16	90
0.35	0.19	0.21	118
0.40	0.21	0.26	146
0.45	0.24	0.33	185
0.50	0.27	0.40	224
0.55	0.29	0.49	275
0.60	0.32	0.58	325
0.65	0.34	0.67	376
0.70	0.37	0.76	426
0.75	0.40	0.84	471
0.80	0.42	0.90	505
0.85	0.45	0.95	533
0.90	0.48	0.98	550
0.95	0.50	1.00	561
1.00	0.53	0.99	555
1.05	0.56	0.96	539
1.10	0.58	0.92	516
1.15	0.61	0.86	482
1.20	0.64	0.80	449
1.25	0.66	0.74	415
1.30	0.69	0.68	381
1.35	0.72	0.62	348
1.40	0.74	0.56	314
1.45	0.77	0.51	286
1.50	0.80	0.47	264
1.55	0.82	0.43	241
1.60	0.85	0.39	219
1.65	0.87	0.36	202
1.70	0.90	0.33	185
1.75	0.93	0.30	168
1.80	0.95	0.28	157
1.85	0.98	0.26	146
1.90	1.01	0.24	135
1.95	1.03	0.22	123
2.00	0.06	0.20	112
2.05	0.09	0.19	107
2.10	1.11	0.17	95
2.15	1.14	0.16	90
2.20	1.17	0.15	84
2.25	1.19	0.14	79
2.30	1.22	0.13	73
2.35	1.25	0.12	67
2.40	1.27	0.11	62

Source: U.S.G.S., 1986

2.1.7 Water Quality Volume and Peak Flow

2.1.7.1 Water Quality Volume Calculation

The Water Quality Volume (WQ_v) is the treatment volume required to remove a significant percentage of the stormwater pollution load, defined in this Manual as an 80% removal of the average annual post-development total suspended solids (TSS) load. This is achieved by intercepting and treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year.

The water quality treatment volume is calculated by multiplying the 85th percentile annual rainfall event by the volumetric runoff coefficient (R_v) and the site area. R_v is defined as:

$$\mathbf{R_v = 0.05 + 0.009(I)} \tag{2.1.20}$$

Where: I = percent of impervious cover (%)

For the state of Georgia, the average 85th percentile annual rainfall event is 1.2 inches. Therefore, WQ_v is calculated using the following formula:

$$\mathbf{WQ_v = \frac{1.2\, R_v\, A}{12}} \tag{2.1.21}$$

Where: WQ_v = water quality volume (acre-feet)

R_v = volumetric runoff coefficient

A = total drainage area (acres)

WQ_v can be expressed in inches simply as $1.2(R_v) = Q_{wv}$

2.1.7.2 Water Quality Volume Peak Flow Calculation

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as for sand filters and infiltration trenches. An arbitrary storm would need to be chosen using the Rational Method, and conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The following procedure can be used to estimate peak discharges for small storm events.

It relies on the Water Quality Volume and the simplified peak flow estimating method above. A brief description of the calculation procedure is presented below.

Step 1: Using WQv, a corresponding Curve Number (CN) is computed utilizing the following equation:

$$\mathbf{CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\, Q_{wv}P)^{1/2}]}$$

Where, P = rainfall, in inches (use 1.2 inches for the Water Quality Storm in Georgia)

Q_{wv} = Water Quality Volume, in inches ($1.2R_v$)

Step 2: Once a CN is computed, the time of concentration (t_c) is computed (based on the methods described in this section).

Step 3: Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the water quality storm event is computed using a slight modification of the Simplified SCS Peak Runoff Rate Estimation technique of subsection 2.1.5.7. Use appropriate rainfall distribution type (either Type II or Type III).

- Read initial abstraction (Ia), compute I_a/P
- Read the unit peak discharge (q_u) for appropriate tc
- Using WQ_v, compute the peak discharge (Q_{wq})

$$Q_{wq} = q_u \times A \times Q_{wv}$$

where Q_wq = the water quality peak discharge (cfs)
q_u = the unit peak discharge (cfs/mi^2/inch)
A = drainage area (mi^2)
Qwv = Water Quality Volume, in inches (1.2R_v)

2.1.7.3 Example Problem

Using the data and information from the example problem in subsection 2.1.5.8 calculate the water quality volume and the water quality peak flow.

Calculate water quality volume (WQ_v)

Compute volumetric runoff coefficient, R_v

$$\mathbf{R_v = 0.05 + (0.009) = 0.05 + (0.009(8/50 \times 100\%)) = 0.37}$$

Compute water quality volume, WQ_v

$$\mathbf{WQ_v = 1.2(RV)(A)/12 = 1.2(0.37)(50)/12 = 1.85\ acre\text{-}feet}$$

Calculate water quality peak flow

Compute runoff volume in inches, Qwv:

$$\mathbf{Q_{wv} = 1.2R_v = 1.2 \times 0.37 = 0.44\ inches}$$

Computer curve number:

$$\mathbf{CN = 1000 / [10 + 5P + 10Q - 10(Q_{wv}^2 + 1.25\ Q_{wv}\ P)\ 1/2]}$$

$$\mathbf{CN = 1000 / [10 + 5*1.2 + 10*0.252 - 10(0.252^2 + 1.25*0.252*1.2)\ 1/2] = 84}$$

$\mathbf{t_c = 0.34}$ (computed previously)

$$\mathbf{S = 1000 / CN - 10 = 1000 / 84 - 10 = 1.90\ inches}$$

$$\mathbf{0.2S = I_a = 0.38\ inches}$$

$$\mathbf{I_a / P = 0.38 / 1.2 = 0.317}$$

Find q_u:

From Figure 2.1.5-6 for I_a/P = 0.317 $\mathbf{q_u = 535\ cfs/mi^2/in}$

Compute water quality peak flow:

$$\mathbf{Q_{wq} = q_u \times A \times Q_{wv} = 535 \times 50/640 \times 0.44 = 18.4\ cfs}$$

2.1.8 Water Balance Calculations

2.1.8.1 Introduction

Water balance calculations help determine if a drainage area is large enough, or has the right characteristics, to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for retention pond and wetland design.

The details of a rigorous water balance are beyond the scope of this manual. However, a simplified procedure is described herein that will provide an estimate of pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

2.1.8.2 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential):

$$\Delta \mathbf{V} = \Sigma \mathbf{I} - \Sigma \mathbf{O} \qquad \textbf{(2.1.22)}$$

Where: Δ = "change in"

V = pond volume (ac-ft)

∑ = "sum of"

I = Inflows (ac-ft)

O = Outflows (ac-ft)

The inflows consist of rainfall, runoff and base flow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 2.1.22 can be changed to reflect these factors.

$$\Delta \mathbf{V} = \mathbf{P} + \mathbf{Ro} + \mathbf{Bf} - \mathbf{I} - \mathbf{E} - \mathbf{Et} - \mathbf{Of} \qquad \textbf{(2.1.23)}$$

Where: P = precipitation (ft)

Ro = runoff (ac-ft)

Bf = base flow (ac-ft)

I = infiltration (ft)

E = evaporation (ft)

Et = evapotranspiration (ft)

Of = overflow (ac-ft)

Rainfall (P) – Monthly rainfall values can be obtained from State climatology data at:

http://climate.engr.uga.edu/info.html

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) it becomes acre-feet of volume. Table 2.1.8-1 shows monthly rainfall rates for Atlanta based on a 30-year period of record at Hartsfield-Atlanta International Airport.

Runoff (R_o) – Runoff is equivalent to the rainfall for the period times the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall. In lieu of gage information, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model. Two other methods have been proposed.

Equation 2.1.20 gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Georgia, this is equivalent of about a 10% runoff volume loss. Thus a factor of 0.9 should be applied to the calculated R_v value to account for storms that produce no runoff. Equation 2.1.24 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$Q = 0.9\ PR_v \quad (2.1.24)$$

Where: P = precipitation (in)

Q = runoff volume (in)

R_v = volumetric runoff coefficient [see equation 2.1.20]

Ferguson (1996) has performed simulation modeling in an attempt to quantify an average ratio on a monthly basis. For the Atlanta area he has developed the following equation:

$$Q = 0.235 P / S^{0.64} - 0.161 \quad (2.1.25)$$

Where: P = precipitation (in)

Q = runoff volume (in)

S = potential maximum retention (in) [see equation 2.1.6]

Table 2.1.8-1 Water Balance Values for Atlanta, Georgia

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation (ft)	0.40	0.40	0.48	0.36	0.36	0.30	0.42	0.31	0.29	0.25	0.32	0.36
Turf Evptr. (ft)	0.06	0.07	0.15	0.27	0.44	0.56	0.61	0.56	0.41	0.25	0.11	0.06

Annual Precipitation (ft)	4.25
Turf Evptr. (ft)	3.55

Source: Ferguson and Debo, 1990 and http://www.griffin.peachnet.edu/

Base flow (Bf) – Most stormwater ponds and wetlands has little, if any, base flow, as they are rarely placed across perennial streams. If so placed, base flow must be estimated from observation or through theoretical estimates. Methods of estimation and base flow separation can be found in most hydrology textbooks.

Infiltration (I) – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation as:

$$I = Ak_h G_h \quad (2.1.26)$$

Where: I = infiltration (ac-ft/day)

A = cross sectional area through which the water infiltrates (ac)

K_h = saturated hydraulic conductivity or infiltration rate (ft/day)

G_h = hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate Table 2.1.8-2 can be used.

Table 2.1.8-2 Saturated Hydraulic Conductivity		
Material	**Hydraulic Conductivity**	
	in/hr	**ft/day**
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.1
Silty clay	0.04	0.08
Clay	0.02	0.04

Source: Ferguson and Debo, "On-Site Stormwater Management," 1990

Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used though there are only two pan evaporation sites active in Georgia (Lake Allatoona and Griffin). A pan coefficient of 0.7 is commonly used to convert the higher pan value to the lower lake values.

Table 2.1.8-3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from five stations in and around Georgia. Figure 2.1.8-1 depicts a map of annual free water surface (FWS) evaporation averages for Georgia based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural stormwater ponds and wetlands being designed in Georgia. Total annual values can be estimated from this map and distributed according to Table 2.1.8-3.

Table 2.1.8-3 Evaporation Monthly Distribution

Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
3.20%	4.40%	7.40%	10.30%	12.30%	12.90%	13.40%	11.80%	9.30%	7.00%	4.70%	3.20%

Evapotranspiration (Et). Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of Et for crops in Georgia is well documented and has become standard practice. However, for wetlands the estimating methods are not documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Values for turf are given in Table 2.1.8-1 based on the Blaney-Criddle method. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating Et only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based Et estimates and a decision made. Crop-based Et estimates can be obtained from typical hydrology textbooks or from the web sites mentioned above.

Overflow (Of) – Overflow is considered as excess runoff, and in water balance design is either not considered, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall- runoff, large storms would play an important part in pond design.

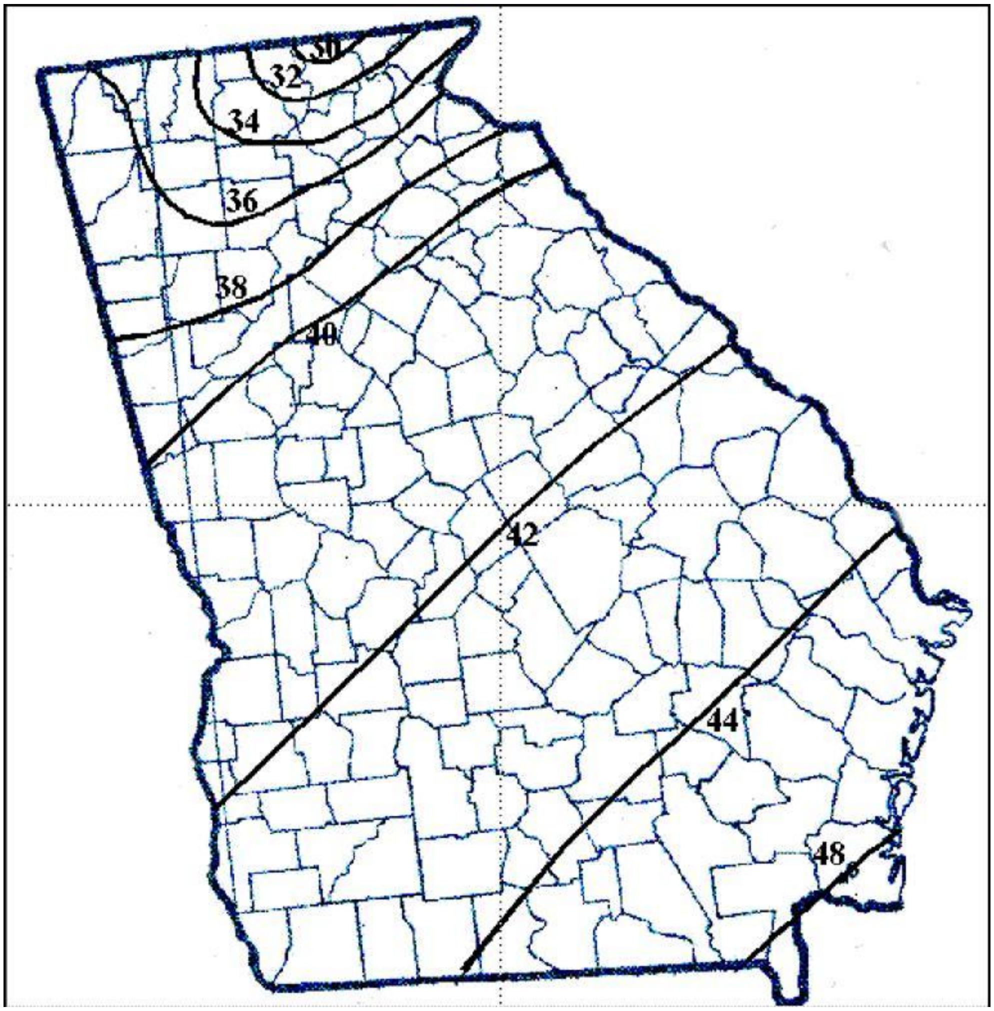


Figure 2.1.8-1 Average Annual Free Water Surface Evaporation (in inches)

(Source: NOAA, 1982)

2.1.8.3 Example Problem

Austin Acres, a 26-acre site in Augusta, is being developed along with an estimated 0.5-acre surface area pond. There is no base flow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data we find that the site is 75% impervious with sandy clay loam soil.

- From equation 2.1.20, $R_v = 0.05 + 0.009\ (75) = 0.73$. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 2.1.8-1 is about 42 inches.
- For a sandy clay loam the infiltration rate is I = 0.34 ft/day (Table 2.1.8-2).
- From a grading plan it is known that about 10% of the total pond area is sloped greater than 1:4.
- Monthly rainfall for Augusta was found from the Web site provided above.

Table 2.1.8-4 shows summary calculations for this site for each month of the year.

Table 2.1.8-4 Summary Information for Austin Acres

1		J	F	M	A	M	J	J	A	S	O	N	D
2	Days/mo	31	28	31	30	31	30	31	31	30	31	30	31
3	Precipitation (in)	4.05	4.27	4.65	3.31	3.77	4.13	4.24	4.5	3.02	2.84	2.48	3.40
4	Evap Dist	3.2%	4.4%	7.4%	10.3%	12.3%	12.9%	13.4%	11.8%	9.3%	7.0%	4.7%	3.2%
5	Ro (ac-ft)	5.70	6.01	6.55	4.66	5.31	5.82	5.97	6.34	4.25	4.00	3.49	4.79
6	P (ac-ft)	0.17	0.18	0.19	0.14	0.16	0.17	0.18	0.19	0.13	0.12	0.10	0.14
7	E (ac-ft)	0.06	0.08	0.13	0.18	0.22	0.23	0.23	0.21	0.16	0.12	0.08	0.06
8	I (ac-ft)	5.01	4.52	5.01	4.85	5.01	4.85	5.01	5.01	4.85	5.01	4.85	5.01
9													
10	Balance (ac-ft)	0.81	1.59	1.61	-0.23	0.24	0.92	0.91	1.31	-0.63	-1.01	-1.33	-0.13
11	Running Balance (ac-ft)	0.81	2.00	2.00	1.77	2.00	2.00	2.00	2.00	1.37	0.36	0.00	0.00

Explanation of Table:

1. Months of year
2. Days per month
3. Monthly precipitation from web site is shown in Figure 2.1.8-2.
4. Distribution of evaporation by month from Table 2.1.8-3.
5. Watershed efficiency of 0.65 times the rainfall and converted to acre-feet.
6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet
7. Evaporation equals monthly percent of 42 inches from line 4 converted to acre-feet
8. Infiltration equals infiltration rate times 90% of the surface area plus infiltration rate times 0.5 (banks greater than 1:4) times 10% of the pond area converted to acre-feet
9. Lines 5 and 6 minus lines 7 and 8
10. Accumulated total from line 10 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design

It can be seen that for this example the pond has potential to go dry in winter months. This can be remedied in a number of ways including compacting the pond bottom, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area.

Station: (90495) AUGUSTA_WSO_AIRPORT From Year=1961 To Year=1990
Missing Data: 0.0%

	Total Precipitation							Snow			#Days Precip		
	Mean	High	--Yr	Low	--Yr	1-Day Max		Mean	High	--Yr	=>.10	=>.50	=>1.
Ja	4.05	8.91	87	0.75	81	2.78	25/1978	0.3	2.3	88	7	3	1
Fe	4.27	7.67	61	0.69	68	3.50	5/1985	1.0	14.0	73	7	3	1
Ma	4.65	11.92	80	0.88	68	5.31	10/1967	0.0	1.1	80	7	3	1
Ap	3.31	8.43	61	0.60	70	2.71	27/1961	0.0	0.0	0	5	2	1
Ma	3.77	9.61	79	1.57	87	4.44	31/1981	0.0	0.0	0	7	2	1
Jn	4.13	8.84	89	0.68	84	2.95	12/1964	0.0	0.0	0	6	3	1
Jl	4.24	11.43	67	1.02	87	3.67	21/1979	0.0	0.0	0	8	2	1
Au	4.50	11.34	86	0.65	80	5.95	29/1964	0.0	0.0	0	6	3	1
Se	3.02	9.51	75	0.31	84	4.55	20/1975	0.0	0.0	0	5	2	1
Oc	2.84	14.82	90	0.01	63	5.32	12/1990	0.0	0.0	0	4	2	1
No	2.48	7.76	85	0.57	73	3.43	22/1985	0.0	0.0	0	4	2	1
De	3.40	8.65	81	0.96	80	2.89	16/1970	0.0	0.4	71	6	3	1
An	44.66	66.04	64	32.96	78	5.95	29/08/64	1.4	14.4	73	71	30	13
Wi	11.71	20.26	87	5.62	86	3.50	5/02/85	1.3	14.4	73	20	8	3
Sp	11.73	19.93	84	4.00	85	5.31	10/03/67	0.0	1.1	80	18	8	3
Su	12.87	24.89	64	7.08	80	5.95	29/08/64	0.0	0.0	0	20	9	4
Fa	8.35	18.50	90	1.96	84	5.32	12/10/90	0.0	0.0	0	12	5	2

Figure 2.1.8-2 Augusta Precipitation Information

2.1.9 Downstream Hydrologic Assessment

The purpose of the overbank flood protection and extreme flood protection criteria is to protect downstream properties from flood increases due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff. Further, due to a site's location within a watershed, there may be very little reason for requiring overbank flood control from a particular site. This section outlines a suggested procedure for determining the impacts of post-development stormwater peak flows and volumes on downstream flows that a community may require as part of a developer's stormwater management site plan.

2.1.9.1 Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 2.1.9-1. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained.

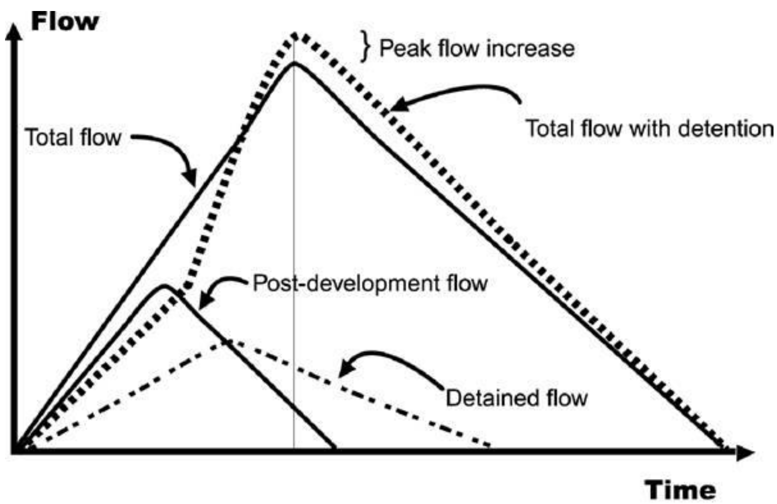


Figure 2.1.9-1 Detention Timing Example

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

Figure 2.1.9-2 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre- development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

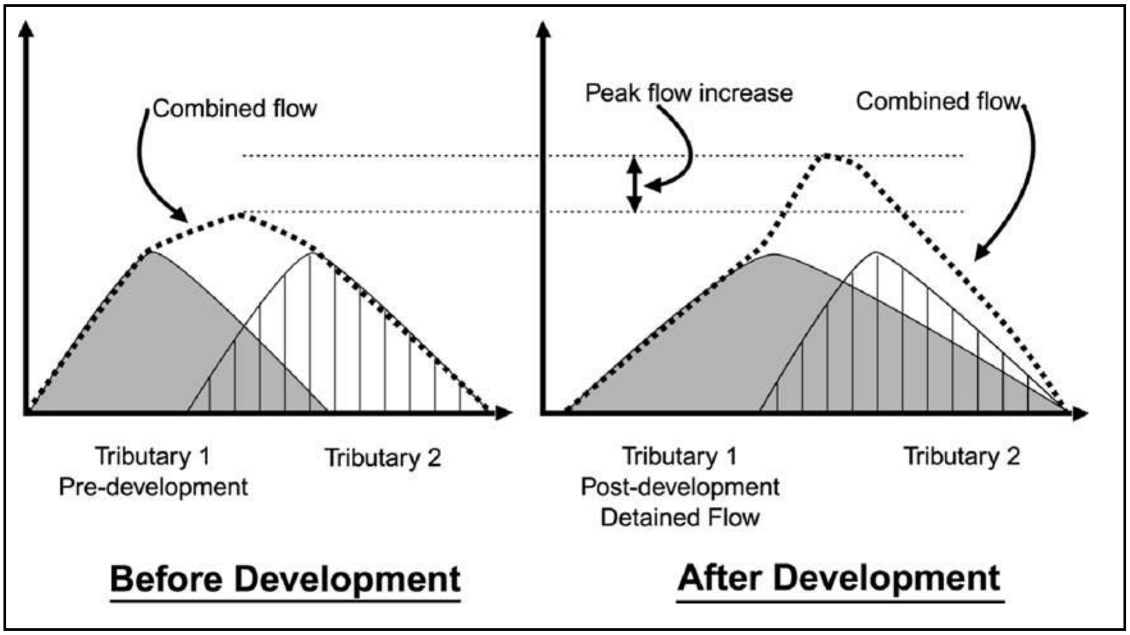


Figure 2.1.9-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

2.1.9.2 The Ten-Percent Rule

In this Manual the “ten percent” criterion has been adopted as the most flexible and effective approach for ensuring that stormwater quantity detention ponds actually attempt to maintain pre-development peak flows throughout the system downstream.

The ten-percent rule recognizes the fact that a structural control providing detention has a “zone of influence” downstream where its effectiveness can be felt. Beyond this zone of influence the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in the application of the ten-percent rule are:

(1) Determine the target peak flow for the site for predevelopment conditions.

(2) Using a topographic map determine the lower limit of the zone of influence (10% point).

(3) Using a hydrologic model determine the pre-development peak flows and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.

(4) Change the land use on the site to post-development and rerun the model.

(5) Design the structural control facility such that the overbank flood protection (25-year) post-development flow does not increase the peak flows at the outlet and the determined tributary junctions.

(6) If it does increase the peak flow, the structural control facility must be redesigned.

2.1.9.3 Example Problem

Figure 2.1.9-3 illustrates the concept of the ten-percent rule for two sites in a watershed.

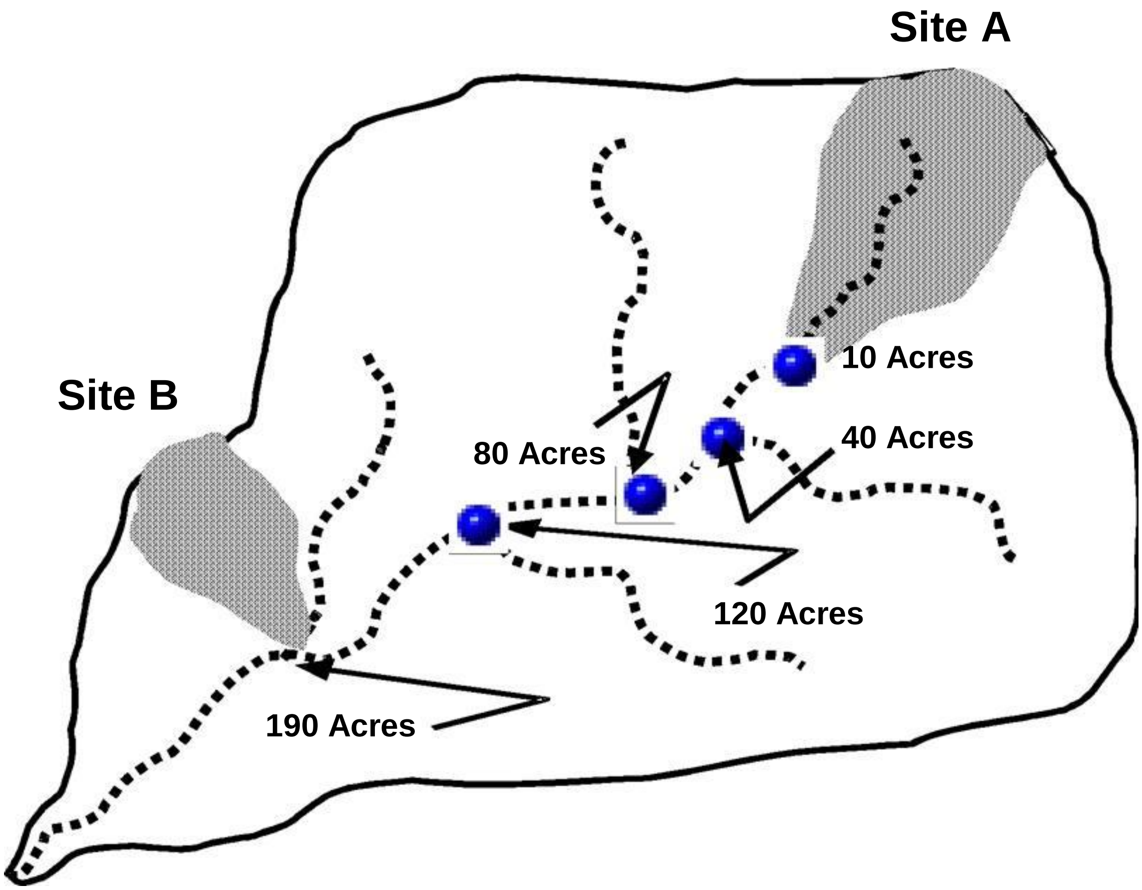


Figure 2.1.9-3 Example of Ten-Percent Rule

Discussion

Site A is a development of 10 acres, all draining to a wet ED stormwater pond. The overbank flooding and extreme flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked “80 acres.” The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase at the 80-acre point then there will be no increase through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single existing condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows that a detention facility, in this case, will actually increase the peak flow in the stream.

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